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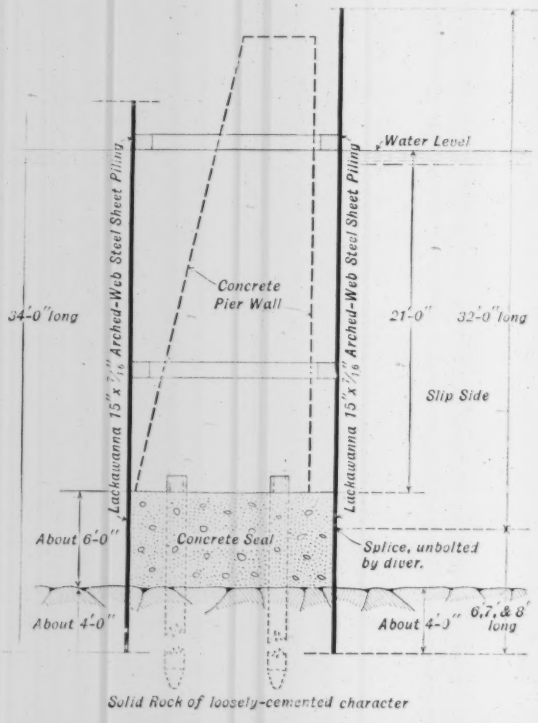

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Engineering News

VOLUME 75 MAY 25, 1916 NUMBER 21

New Water Tunnel at Chicago

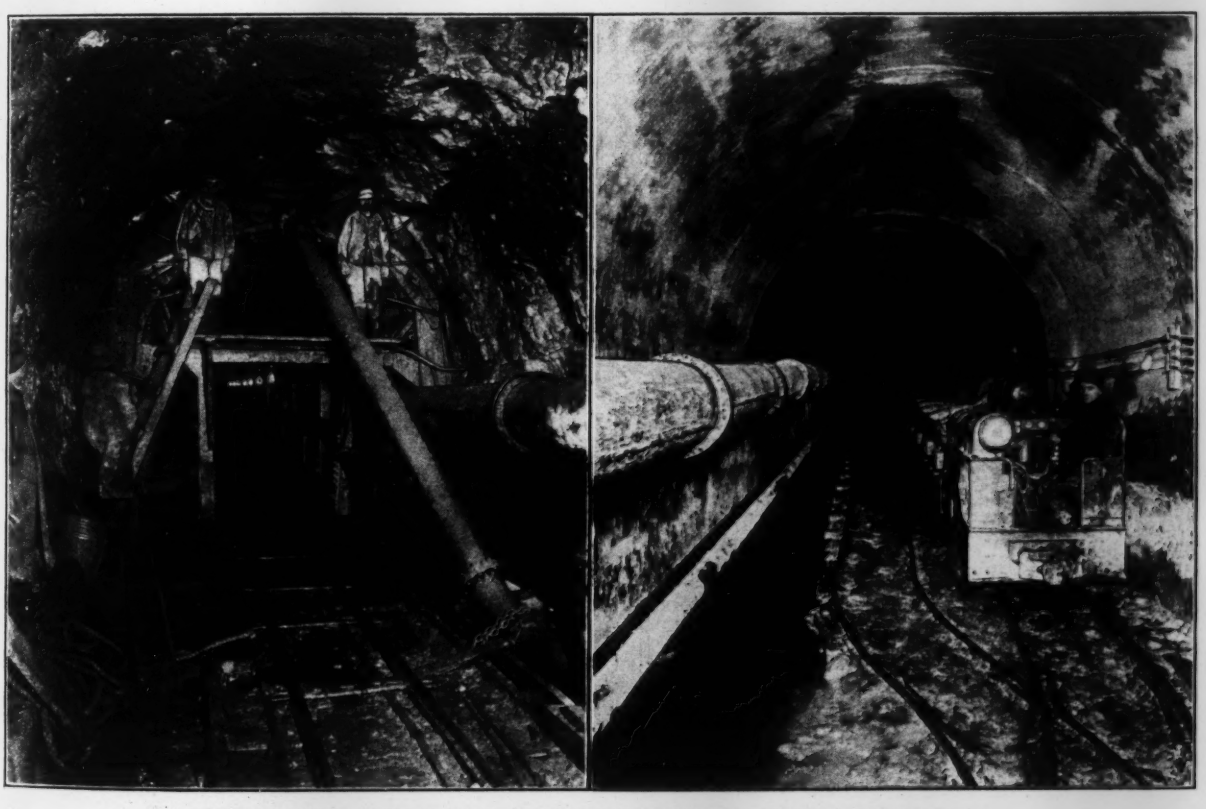
SYNOPSIS—New intake tunnel for Chicago avoids danger of soft ground by going deep enough to strike sound rock. Built by city day-labor. Top-heading system worked on two-shift plan found best; rates of advance up to 22 ft. per day. Economy in handling spoil accomplished by screening out fine rock in the tunnel and using it for immediate mixing and placing of lining concrete, so that only the oversize rock goes out of the tunnel. Concrete blown into lining with pneumatic mixer and conveyor.

City day-labor is producing rapid progress and low cost in the construction of the Wilson Ave. intake tunnel of the Chicago water-supply system. Several ingenious features of the plant equipment and methods of working, devised by the city's engineers and executed under their direction, are responsible for these results.

The tunnel will be about 8 mi. long; of the total length under the lake, half is from the shore shaft to an intake shaft and crib. The land portion extends under Wilson Ave. to a new pumping station serving the northwest section of the city. It has also a short branch tunnel to furnish additional supply to the present Lake View pumping station. The main tunnel is 12x12 ft. and 13x13 ft., with vertical sides, semicircular roof and invert floor. The flow capacity is 350,000,000 gal. per day. The tunnel was described in *Engineering News*, Apr. 22, 1915. Figs. 1 and 2 are views in the drift and completed tunnel under the lake.

To keep the entire tunnel in solid rock, it was placed at an unprecendented depth—160 ft.—the prior intake tunnels having encountered many construction difficulties due to soft ground. Diamond-drill borings were made along the line before deciding on the depth of the tunnel.

Besides the intake shaft, shore shaft and pumping-station shaft, there are two intermediate working shafts



FIGS. 1 AND 2. THE WILSON AVE. LAKE TUNNEL—ROUGH AND LINED

At left, 8-in. blow pipe from MacMichael concrete mixer going through dam over Blaw form section. Run-of-screen aggregate is used, up to 4-in. size, without sand addition, yet a remarkably even surface is obtained, as seen at right. The upper pipe in both views is the sheet-steel ventilation pipe, jointed with rubber-packed loose flanges, and so light that a 20-ft. length can be handled by two men. The clear air in both views is due to excellence of ventilation and absence of leakage except near face

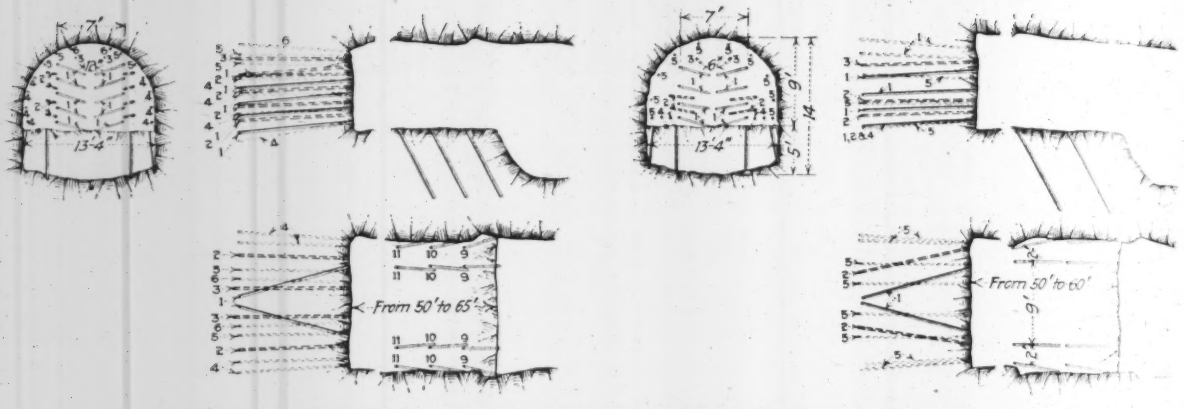


FIG. 3. ALTERNATIVE METHODS OF DRIVING THE TUNNEL ON THE TOP HEADING SYSTEM

on the land portion of the tunnel. A contract was let for the intake end of the work, as this included the steel crib and the city did not wish to acquire the floating equipment necessary for work in the lake.

The design and execution of the work are under the direction of John Ericson, City Engineer. The construction is in direct charge of Henry W. Clausen, Engineer of Water-Works Construction, who is practically the general superintendent of all construction work for the water-works. The following description is prepared from a paper read by Mr. Clausen at a meeting of the Western Society of Engineers, at Chicago.

Organization of Forces

The work at each shaft is under an assistant engineer and a tunnel foreman, the latter reporting to the engineer. The heading foreman (in charge of drilling and blasting), the mucking boss, the concrete foreman, the track-gang boss, the hoisting enginemen and such unattached labor as top laborers and pumpmen all report to the tunnel foreman.

Union labor is employed at the union scale of wages, which is higher than for tunnel work in other parts of the country. The lowest pay is $4.40 per day, for muckers. Drillers and miners get $5; skilled mechanics, $6, with overtime.

Headhouses and Shaft Equipment

At some shafts, where space was available, high headhouses and trestles were constructed, so that all muck (stone) might be stored. At other shafts the structures were only high enough for dumping into bins from which the stone was removed daily.

The power used is 220-volt alternating current, transformers being placed to furnish as much as 800 hp. The cost of current averages 13c. per kw.-hr. The Thomas electric hoists are driven by 50-hp. slip-ring motors and give a speed of 200 ft. per min. for 5,000-lb. loads. It is thought that still better results could be obtained by adopting the double-skip method.

Compressed air for the drills and concreting apparatus is furnished by three or four motor-driven Ingersoll-Rand compressors, having a capacity of 600 cu.ft. of free air per min. delivered at 110 lb. pressure. A 7-in. header leads to a vertical receiver 4x12 ft.; from this a 6-in. pipe is carried down the shaft and branches to 4-in. lines in the short drifts, while a 6-in. line is used in the long drift under the lake.

To determine the most efficient drills for the work, different kinds were used at different shafts. The old piston drill and water drill are found to be the most economical in repair; the water drill is the faster (though the difference is not marked) and uses less air. The drills at the face are mounted on both columns and heading bars. The former method seems to have the advantage. Bench holes are drilled from the column or bar as much as possible.

For handling the muck a double-track line of 24 in. gage was laid, having 20-lb. rails and steel ties, with about two wood ties to 20 ft. of track to prevent the steel ties from creeping. Steel side-dump cars of 1-yd. capacity are used.

When the drifts became too long for mule haulage, 5-ton Baldwin gasoline locomotives were used, having four-cylinder 40-hp. engines. They can run at 8 mi. per hr. and handle usually 20 to 25 loaded cars, but have handled as many as 57 cars. The exhaust is passed through a water box, and ordinarily the escaping gases are sufficiently diluted to be unobjectionable.

Ventilation and Drainage

An exceptionally large ventilating capacity is provided and has helped materially in the rapid progress by quickly removing the smoke and gases after blasting. For each drift up to 6,000 ft. in length, there is a centrifugal compressor with a capacity of 5,000 cu.ft. per min. at 1.5 lb. pressure. Longer drifts have two compressors in series, giving the same capacity but at 3 lb. pressure.

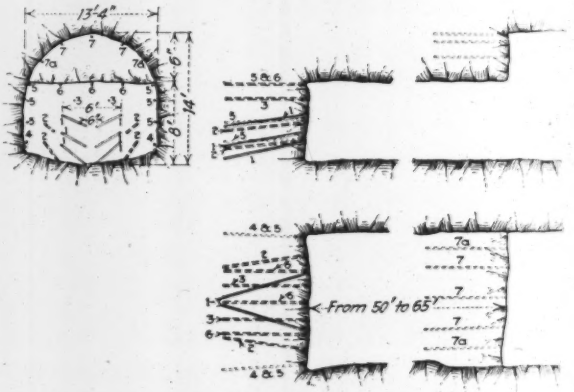


FIG. 4. METHOD OF DRIVING THE TUNNEL WITH A BOTTOM HEADING
Used at one shaft only

Each compressor has a 50-hp. motor. The air is led to within about 100 ft. of the bench in an 18-in. pipe suspended along the side of the tunnel, over the drainage ditch.

The pipe was designed and made specially for this job. Lightness was the controlling consideration. It is of sheet steel, asphalt-coated; the 20-ft. lengths have ends flanged back and fitted with loose malleable-iron flanges. Soft-rubber gaskets are inserted. The pipe is so light that two men can easily carry a 20-ft. length, which greatly facilitates the moving of the line at concrete forms, etc.

The ventilating system can be used to force the air in or to exhaust the air and smoke after blasting. A bypass with valves provides for this without reversing the compressor. In winter the air blown in is heated.

For drainage two sizes of motor-driven centrifugal pumps are used, to work against heads of about 150 ft.; they are of 1,000 and 300 gal. capacity, with 75- and 20-hp. motors. The tunnel is almost entirely dry, and such water as is encountered enters through the sides, at 4 or 5 ft. above the bottom. No water enters at the roof.

To determine which method of tunneling was the most economical and speedy and best adapted to Chicago conditions, it was decided to try different ones. The standard method of top heading and bench, operating on the two-shift basis, was used at the short shaft and two intermediate land shafts. This method (Fig. 3) involves the drilling of from 28 to 34 holes in the heading and 8 to 12 holes in the bench. The cut holes are from 12 to 14 ft. deep, and some 15-ft. steels were made up in order to increase the shot holes. Alternative methods of driving by the top-heading system are shown in Fig. 3.

The bottom-heading method with breakdown roof, on the three-shift basis, was used in the single heading from one shaft, where the territory was not so built up, and it was found that no complaints resulted from the blasting operations being carried on during all hours of the day and night. This method (Fig. 4) involves the drilling of about 23 holes in the bottom heading and 5 holes in the roof heading. The cut holes are from 9 ft. to 10 ft. deep.

At the latter shaft an experimental shooting platform was constructed for the purpose of intercepting the rock shot down from the roof, thus avoiding blocking the free passageway to the bottom heading for a certain time while the rock from the roof was being mucked out. The platform was constructed of steel and lasted but a short time, finally becoming so badly bent as to make it of no further use and it was abandoned.

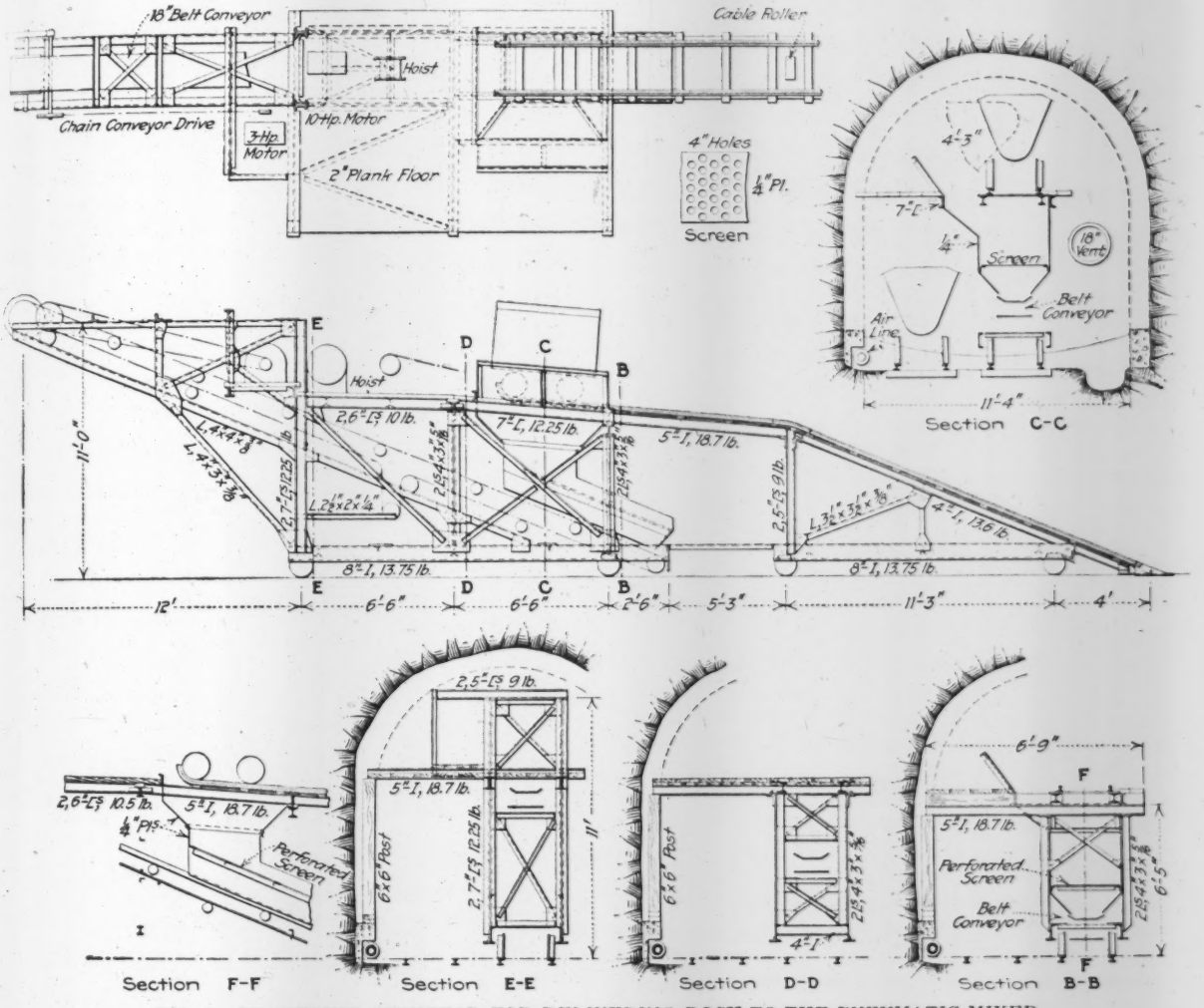


FIG. 5. TRAVELING CONVEYOR FOR DELIVERING ROCK TO THE PNEUMATIC MIXER

In general the top heading and bench method is the most economical, operated on the two-shift basis. If there is any quantity of water, it can escape freely instead of being blocked by the breakdown shot of the top heading. The second and most important reason is that on the two-shift basis a certain task can be given to the gang, which must be completed before they leave the work. It is the duty of the miners to set up columns,

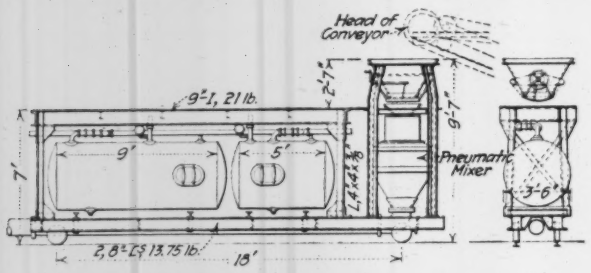


FIG. 6. TRAVELING PNEUMATIC CONCRETE MIXER

drill and load their holes, and shoot their rounds before going home. Likewise it is the duty of the muckers to muck out all the rock from the previous shift. It is understood that the time of the shift ends whenever these tasks are completed.

With a well-organized crew, for a 10- to 12-ft. round, the muckers will complete their work in from 6½ to 7 hr. and the miners in from 7 to 7½ hr. If the gang is not up to standard, it may take from 9 to 10 hr., for which they only receive one day's pay. The fact that a

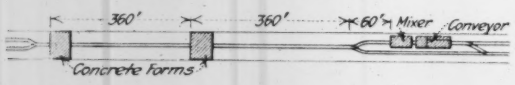


FIG. 7. PLAN SHOWING ARRANGEMENT OF FORMS AND CONCRETING PLANT

shift can be held responsible for a definite amount of work spurs each man to do his part.

In the three-shift method, the gang only works 8 hr. and quits, the succeeding shift taking up the work where the previous shift left off. If the previous gang neglected their work, it is very much harder to place this responsibility without argument and question. This latter fact is of extreme importance when (as in this case) the work is under the civil-service system, because absolute evidence must be forthcoming before a man can be discharged.

The total daily progress was not any greater by the three-shift method, although a greater number of men was employed.

Concrete Mixing and Placing Plant

The concrete is mixed and placed by the MacMichael pneumatic process. The mixing is done in the tunnel itself. The plant includes a conveyor traveler and a concreting traveler, both built of steel (Figs. 5 and 6). The former has an inclined track up which the car loaded with rock is hauled by a cable and 10-hp. electric hoist. The car is dumped at a screen plate having 4-in. circular holes, and the rock passing through falls upon a belt conveyor which delivers it directly to the mixer hopper. The conveyor is driven by a 3-hp. motor and runs at about 150 ft. per min.

The concreting traveler, shown also in Fig. 9 carries the pneumatic mixer and two reserve air tanks of 14 cu.ft. capacity; these are to prevent sudden drops of pressure in the air-supply line due to the large volume of air required for each discharge of the mixer. An 8-in. pipe leads to the forms.

The plant, comprising the two travelers, is moved 700 ft. every 12 days, under an arrangement of work as sketched in Fig. 7. As the forms approach the mixer, the 8-in. pipe is shortened until another shifting of the plant becomes necessary.

The mixer is leased from the Concrete Mixing and Placing Co., of Chicago. The screening and mixing outfit was constructed by the Stephens-Adamson Manufacturing Co., of Chicago, from the engineers' design.

Placing the Concrete Lining

The method usually employed by contractors in tunnels of this size has been to complete the drift before starting the lining of the tunnel, bringing all material to the surface. After the mining was completed, the excavated rock stored on the surface was crushed and screened and made into concrete, which was brought into the tunnel and placed by hand behind forms of steel or wood. From 30 to 35 ft. per day was lined in this way. When the drift was long, the concrete sometimes had its initial set before arriving at the forms. By a radical departure from this method, the Wilson Ave. tunnel secures a great gain of economy.

It was noticed that the muck or excavated rock contained considerable fine material and would make good concrete. Various compressor foundations and other miscellaneous concrete work on top were built with this material. Test cubes also showed satisfactory strength.

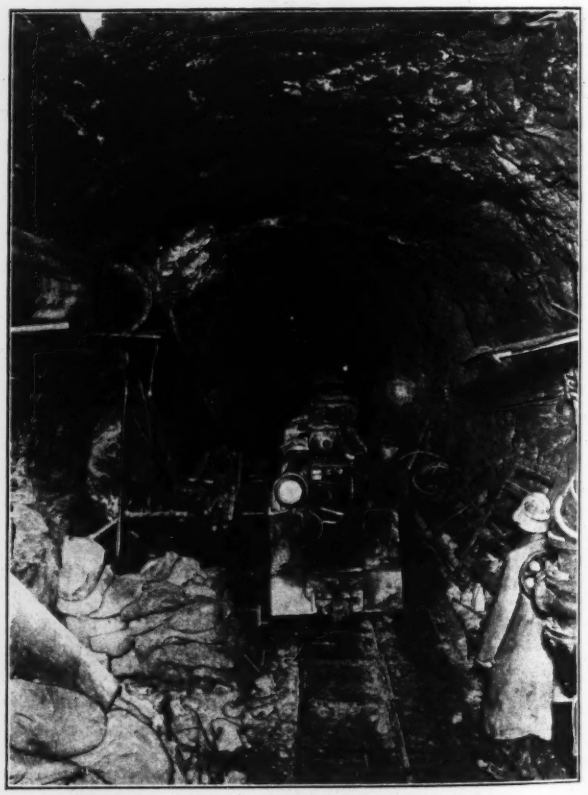


FIG. 8. GASOLINE LOCOMOTIVE AND MUCK TRAIN IN WILSON AVE. LAKE TUNNEL

The rolling stock has automatic couplers and roller bearings

It thus developed that the rock could be used for concrete without previous crushing.

Realizing the possibility of saving time and also of saving the expense of hauling back and forth the material required for the lining, as well as the cost of purchasing crushed stone and sand for the concrete, a method was devised for carrying on both mining and lining simultaneously. For this purpose the screening and mixing machine was installed in the tunnel. It handles all the rock coming from the heading and delivers all rock under 4 in. to the pneumatic mixer. The oversize is rejected and loaded by hand into a car.

The pneumatic mixer delivers the concrete through an 8-in. pipe placed behind the steel forms at a distance of as much as 800 ft. Fig. 1 shows the end of this pipe leading up to the arch. The screening machine and the concrete mixer operate on one track and within the clearance of the final tunnel dimensions.

The forms, Fig. 1 are made of the traveling type. As the mining and lining had to be carried on at the same time, room had to be provided through the forms for transportation of muck to the shaft, with sufficient clearance for the locomotives. These are Blaw forms, constructed of structural shapes and steel plates in 30-ft. sections, and are mounted on roller-bearing wheels running on 70-lb. rails.

By means of this system of working, approximately 60 ft. is lined in two shifts. The net results are that 20 to 22 ft. a day is mined and approximately 60 ft. of tunnel is lined with the excavated material on each working day in each drift.

In advance of the steel forms, concrete guide walls are constructed by hand. Wood forms are used for these, set to true line and grade from the engineer's center line and grade plugs. These concrete walls guide the steel forms so that the finished lining is absolutely true to line and grade. Mr. Clausen states that this is something it was difficult to get a contractor to do, his policy being rather to fit the hole as blasted as near as possible and thus avoid trimming. This would sometimes result in the tunnel being a series of short tangents varying 2 to 6 in. from a straight line.

The Rock-Crushing Plant

At the Lawndale shaft the rock is of a different texture, 50% breaking over 4-in. size. As at this shaft the total excavation was required for concrete, it was decided to put in a crusher. Accordingly, a concreting machine is placed in each drift, and the oversize from the east machine (at the face) is put through the crusher and sent to the west machine in the completed drift.

The crusher was mounted on timbering in an enlarged chamber at the bottom of the shaft. Above it and at one side was a track on which rock cars were run from the shaft cage and dumped directly into the crusher. Beneath was another track for cars to receive the rock.

The disposal of excavated material was a serious problem, particularly at the shore shaft, where there was no available space for storage. The city finally made a contract with the Peoples Crushed Stone Co., which agreed to take the stone away as fast as produced from the shore shaft and to remove the rock piled at the other shafts within a reasonable time after the completion of all the work. The contract provided that crushed stone for the lining of the tunnel should be furnished to the city upon requisition from time to time. The city was thus relieved of considerable expense in the removal of over 400,000 cu.yd. of material. The company erected a crushing and washing plant capable of handling between 750 to 1,000 cu.yd. per 10-hr. day.

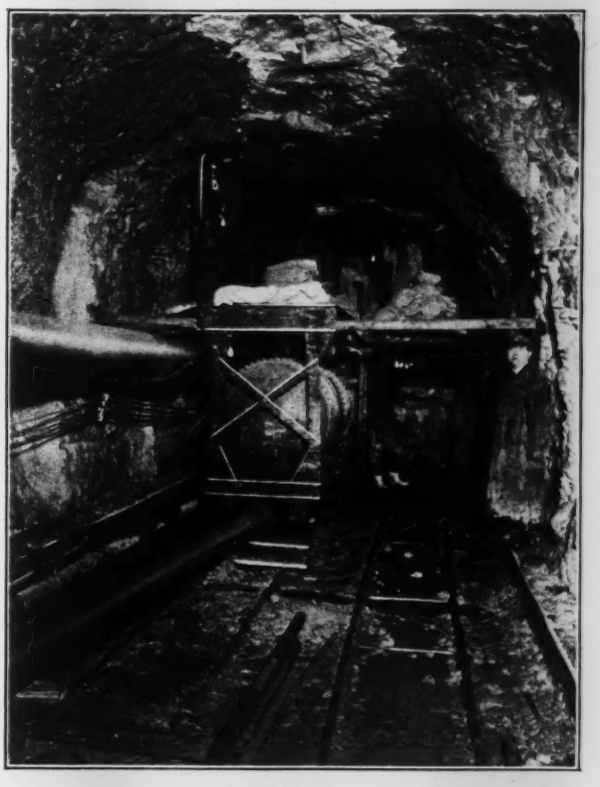


FIG. 9. REAR OF CONCRETING MACHINE IN TUNNEL

Progress and Cost

The average daily tunnel excavation is about 20 ft.; on many days as high as 22 and 23 ft. is made. The maximum has been about 26 ft. One reason for the remarkably rapid advance is the rapid clearing of smoke by the ventilation system. On similar work done by contract, the best daily average was 8 ft. A record progress of 15 ft. on one contract left so much trimming to be done afterward that the average daily rate was lower.

When excavating rock at 20 ft. per day, a round trip of the cage is made every 56 sec., 7 sec. being used each at top and bottom for loading a car. The descent would be made in 10 sec. At the bottom of the Lincoln Ave. shaft, in order to facilitate this operation, there is a spare track on one side of the cage to handle empties.

The cost of excavation of 12-ft. tunnel (labor and material) averages $32.94 per ft. The lining was estimated at $15, but is costing less. For all work in 1915, including depreciation of plant, engineering, inspection, excavation, lining and cleaning, the average cost was $61.48 per ft. The cost by contract was estimated at $75 per ft., and on the basis of the lowest bid it would have been $68.20. Contractors considered the time limit of 42 months for the lake tunnel too short, but Mr. Clausen expects to finish the work within 30 months from the time of starting. In fact, it is expected to finish the lake and land tunnels in February and March, 1917, respectively, or 12 months ahead of time.

Failure of Diversion Dam on Salt River Project

SYNOPSIS—Low weir on gravel foundation above Roosevelt dam fails under overtopping flood. Apron, not of sufficient thickness to withstand erosion, wore down to thin section. Report by board of inspecting engineers.

On Jan. 28 of this year the greater part of the diversion dam at the head of the power canal 19 mi. above the Roosevelt dam on the Salt River project of the United States Reclamation Service went out under extreme flood. The dam was in an isolated location where observation of its behavior was necessarily limited, so extensive information as to its condition and its failure is not available. Inasmuch as the dam was one of the much discussed porous-foundation type, such facts as are obtainable regarding its failure should be put on record.

The Roosevelt diversion dam was built in 1906 across the Salt River, some 20 mi. above the Roosevelt dam, then under construction. It is at the head of the power canal leading down to the main dam and was used during the building of that dam to generate power for the construction machinery. Since that time the canal has been continued in service for power purposes, the current generated being sold in the vicinity of Phœnix and Globe. In the future it is intended to be used as an emergency feeder to the Roosevelt power house.

The details of the dam, as shown on the design drawings, are given in Figs. 1 and 2. On those drawings are also shown such modifications of the design as were made during erection. As will be noticed, it consists of a rubble-concrete ogee-section weir or dam 12 ft. high, 18 ft. wide and 400 ft. long, extending between rock-filled buttressed abutments. It is founded on a porous foundation into which penetrate upstream and downstream cutoff walls, 24 in. thick, and 18-in. transverse walls spaced every 10 ft. It is backed with a loose rock-fill, and the upstream floor is designed to be paved with large boulders. As mentioned below, however, in construction these boulders were paved with an additional concrete floor.

The failure of the dam is best described by the board of engineers appointed by the Reclamation Service to investigate the matter. This board, composed of D. C. Henny, L. C. Hill and William S. Cone, all of whom have an intimate knowledge of the site and of the design, reported in part as follows:

The present condition of the dam is as follows: From sluiceway wall northward, there are 17 ft. in place; 230 ft. washed away, 45 ft. moved downstream 4.5 ft. and tipped back, 8 ft. broken, but nearly in position, and 100 ft. in place at the north end. The water face of the overflow portion of the dam shows heavy wear for a distance of about 2 ft. below the top.

The sluiceway and headgate works are intact. The north abutment has a crack running downward, starting from a contraction joint at about the middle of its river-front face, but deviating in its downward course at an angle of about 10° in a downstream direction. This crack is open at the top about 1½ in., and at the bottom it is nearly closed. The downstream toe wall of the north abutment is washed away, also the earth-fill that was inclosed within the abutment walls; also most of the downstream-bank riprap.

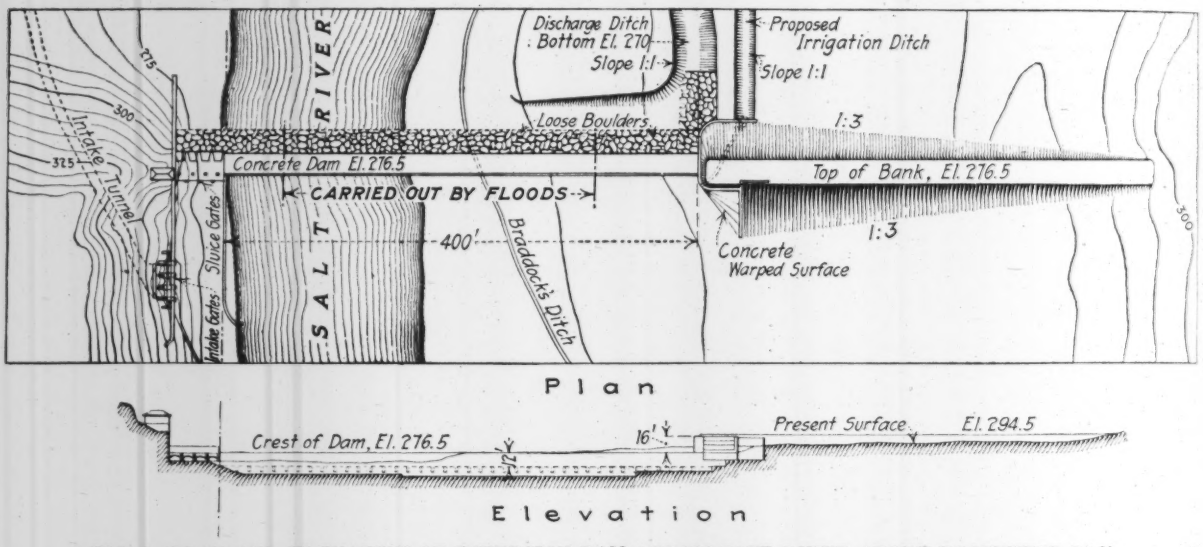


FIG. 1. PLAN AND ELEVATION OF DIVERSION DAM ACROSS SALT RIVER ABOVE ROOSEVELT DAM

The first flood peaked at about 1 p.m. on Jan. 19 at 108,000 sec-ft., when the north-abutment earth dam was slightly overtopped and earth was washed away from back of the abutment without, however, causing a gash or break. The second flood peaked at about the same amount at 6 p.m. on Jan. 28, when the damage to the main structure took place.

The cause of the damage cannot be definitely ascertained. It may be due to wear over the concrete spillway apron from gravel, cobbles and boulders. A small breach in this protection would quickly wash a deep hole that may have undermined the downstream toe of the dam, which rested on gravel. This seems the most plausible explanation. Piping under the dam may have been a momentary feature of destruction, but can hardly be considered the original cause.

It has not been possible, due to continued heavy river flow, to make any accurate survey or determine the depth to which the river channel has cut, the river flow being at present about 3,000 sec.-ft. It is probable that the receding flood will fill, with heavy gravel and sand, deep holes and pockets that may now exist and that the principal work of repair will consist of building of coffer-dams, reconstruction of masonry and

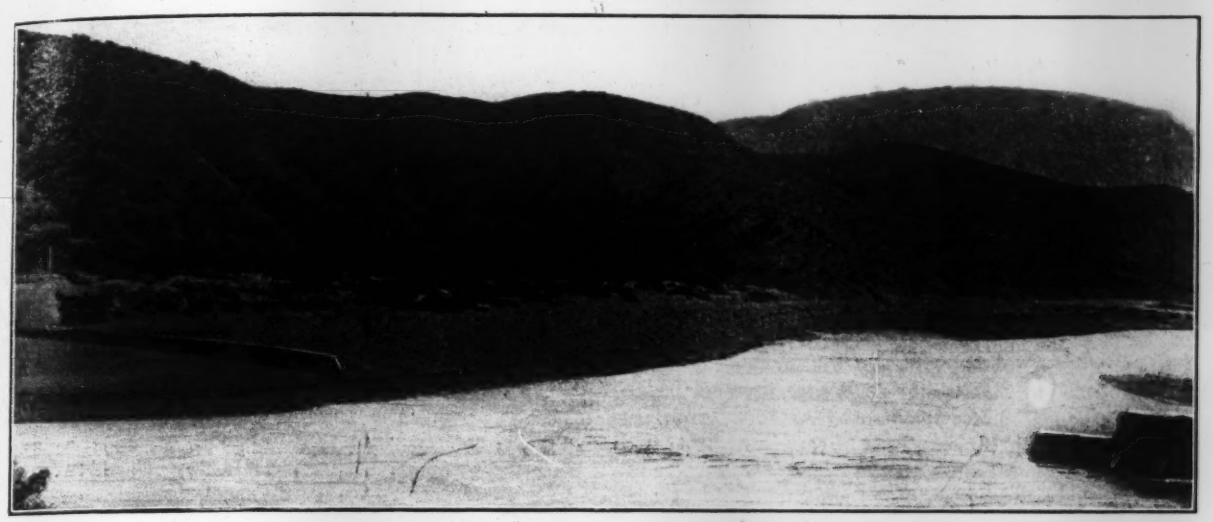


FIG. 3. SITE OF DIVERSION DAM AFTER FAILURE OF JAN. 28, 1916

refill of earth pack of the north abutment. Roughly speaking, 290 ft. of overflow dam and spillway will have to be rebuilt.

In response to further inquiry regarding the dam, L. C. Hill, of the inspecting board and formerly in charge of the projects, writes as follows:

This dam was built under the direct supervision of O. T. Ready, who was the engineer in charge, and the plans for the dam were drawn by A. L. Harris. It was built during the season of 1905, when very heavy floods prevailed in Salt River. The plan that was made up in the office apparently was not exactly the plan followed in the field. To just what depth the two cutoff walls above and below were finally built, I am not now sure. The intention was, however, that these cutoff walls should go down at least 10 ft. below the normal bed of the river in the middle of the stream. In some places it probably went lower than this. At one place in the stream bed there was rather soft material encountered, much softer than in any other part, and here there were a number of piles driven down to depths considerably greater than it was possible to carry the concrete cutoff walls. The apron, which shows on one drawing as being made up of loose rock without any covering, actually was made up of loose rock with a curtain wall at the lower end, and the rock covered on top with about 12 to 18 in. of concrete poured in squares whose dimensions, as I now recollect them, were about 8 ft. on a side. These squares were not absolutely separate. The corners were left joined together.

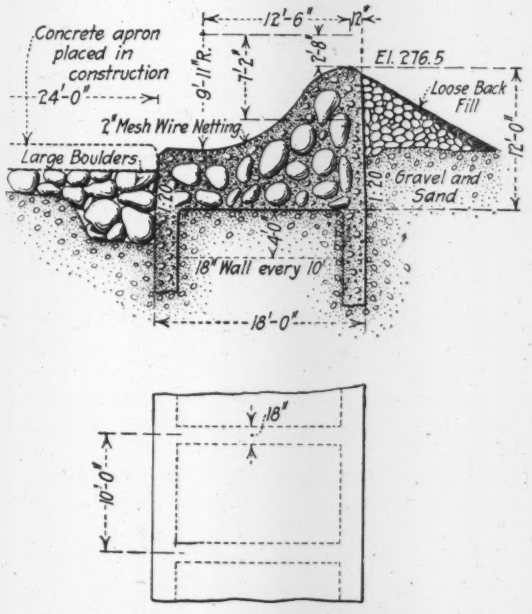


FIG. 2. SECTION THROUGH MAIN WEIR WHICH WENT OUT

I recently inspected this dam with other members of a board. We made inquiries from some young engineers who had happened to be up there some time before either of the large floods had passed over the dam. From talking with these young engineers I learned that the apron seemed to be very badly worn, as they found from walking across the dam on the apron in their bare feet, when there was about a foot or two of water on the apron, that the apron was full of holes from 6 to 8 in. deep. The original apron was left almost smooth, although there was a layer of smooth river boulders worked into the concrete apron, but in no case probably standing above the ordinary level of the apron more than a ½ in. at any point. We examined the upstream face of that part of the dam which is in place, and the concrete looked as if it had been hammered with a sledge hammer for about 2 ft. below the top of the wall. There were evidences that large boulders of 1 ft. to 2 ft. or more in diameter had passed over the dam. The probable cause of failure, although of course not the certain cause, was the wearing out of the apron from the heavy impact of rock carried by the successive floods. No repairs had been made to the face of this apron since it was completed some ten years ago.

There has never been at any time any report of agitation of the water as far down as the end of the apron. From reports made by engineers during the time the Roosevelt dam was building, it appears that when the water reached the depth of 7 ft., more or less, there ceased to be any agitation below the dam, the water passing over in a smooth curve. The greatest agitation occurred with somewhere between 5 and 6 ft. going over the dam, according to reports made several years ago. The fact that the dam had gone through a great many floods of quite considerable magnitude before the big floods of 1916, without any damage to the dam, seems to show that piping was not the primary cause of the failure of the dam. The fact also that it passed successfully through the first and much the larger flood would seemingly corroborate the belief that the failure of the dam was due to rocks breaking up the apron and hence leaving the material just below the toe of the dam exposed to the erosion of the overpouring water. Of course, as soon as this occurred, the dam would fail within a very short time.

The left end of the dam for perhaps 5 or 10 ft. was founded on rock. No other rock appeared at any point along the line of the dam; and the dam was designed for a boulder, sand and gravel foundation and would, I believe, be standing today if the apron had been made of sufficient thickness to sustain the long-continued erosion or if proper repairs had been made to the apron when it was discovered that it was wearing thin or holes were being ground in it.

As stated in the first part of this article, present and future use of the power canal is only that of an emergency feeder to the Roosevelt power house and the failure of the dam at this time has not in any way interfered with the operation of the power plant or of the irrigation systems in Salt River Valley.

Work Done by the U. S. Forest Products Laboratory

Experimental work carried on at the Forest Products Laboratory, maintained by the Forest Service of the United States Department of Agriculture in coöperation with the University of Wisconsin at Madison, Wis., is of direct interest to all industries that depend upon wood in any way. Securing a better and more economical use of wood in all its forms is the object of the laboratory's work. Some of the recent accomplishments are outlined in the following paragraphs. These investigations are for the most part only definite steps in a larger program.

A humidity-regulated kiln has been devised (patented and dedicated to the public) of service in reducing the time for drying—particularly of green wood from the saw. Green white fir has been successfully kiln dried in 43 hr. Unusual and uneven shrinkage of Western woods, notably red cedar, was found to be due to collapse of cell walls during drying. For this wood it was found sufficient to keep the temperature below 135° to 140° so long as the cell walls were wet. The use of sodium fluoride to prevent sap stain on lumber has been successfully developed. This solution also tends to prevent rot.

The consumption of treated wood blocks for paving has fallen off 30% since 1911, owing to trouble from exudation of oil and swelling of the blocks. The Laboratory has succeeded in removing the defects by applying a preliminary and final vacuum in the course of the preservative treatment. The new methods have been adopted by the Committee on Paving of the American Society of Municipal Improvements and by several cities. Better methods of treating piling are now possible to reduce destruction by marine borers. The preservatives are valuable in the following order: Coal-tar creosotes, water-gas-tar creosotes, and wood tars and creosotes. The addition of metallic salts increases the protection. It has been found that creosoted wood can be used in silos without contaminating the ensilage.

Some 70 species of hard wood and 40 species of conifers have been tested (in jars) to determine relative resistance to decay. The junipers and arbor vitæs proved highly durable—Eastern arbor vitæ losing only 7.7% in 12 mo. Resinous hard pines varied from 17 to 58% loss. Sapwood of all conifers except Western red cedar, Western juniper and arbor vitæ lost 60% or more. Great variation has been found in the toxic properties of wood preservatives, showing need of standardization of products. Fungi have been found to increase their resistance to toxic agents, the concentration needed to kill *fomes annosus* rising from 1% to 3% in four generations subjected to preservatives.

An exceedingly unsanitary condition (as to rot fungi) has been found to exist in many lumber yards in the handling of timber, causing local outbreaks of rot in wood buildings. Few precautions are taken to maintain stocks in sound condition or not to sell infected stock to the building trades.

Fireproofing paints for wood have been tested in an experimental firehouse. Of 18 outside paints, five had retarding properties, the retardation being from 1 min. 3 sec. to 2 min. 10 sec. in place of 43 sec. The Laboratory has developed a fireproofing compound that will be patented and dedicated to the public.

Tests of 164 M.ft. b.m. of lumber from [illegible] trees cupped for turpentine (cup and gutter system) showed no difference in the quantity of lumber produced and only ½ to 1% reduction of value.

A drum barker for paper-mill wood has been developed that reduces the waste to 10%. It has been found possible to work this bark up into pulp for low-grade box board. A test instrument has been devised to measure the tearing strength of paper.

In working upon the destructive distillation of hard woods temperature-control apparatus has been devised by which the yield of acetate and alcohol has been increased up to 50%. Ethyl alcohol yield of larches and spruce under digestion has been increased 23%.

Method and Cost of Bitulithic Paving, Pierce Co., Wash.

By David H. White*

This road, paved as described in this article, begins at Alderton, about 15 mi. southeast of Tacoma, Wash., is 5,319 mi. long and ends at Orting. The pavement was laid over a gravel road on alluvial soil. The original grade of the road was utilized as nearly as possible in order to get the benefit of the packed surface for the foundation. The pavement has a 2-in. bitulithic top, 16 ft. wide, containing 49,878 sq.yd., laid on a 5-in. 1:3:6 concrete base, with 6-in. concrete curbs. The number of square yards in the base and curbs is 52,979. The

FIG. 1. SPREADING SAND FOR SEAL COAT

curbs are monolithic with the base. Three-foot bank-run gravel shoulders make the total width of the roadway 23 ft.

The contract was let to the Washington Paving Co. for $76,549, with a 5-yr. maintenance bond costing $500 extra. This made the cost to the county $1.445 per sq. yd., divided as follows: Subgrade and extras, including curbs, 23.4c.; concrete base, 60c.; bitulithic top, 68.6c. The cost per mile was $14,392.

Sand for the bitulithic top and sand and gravel for the concrete base were obtained from a pit owned by the contractors, located about 0.6 mi. from McMillan, where the bitulithic plant was stationed, being 2.2 mi. from one end of the pavement and 3.2 mi. from the other. The average haul for the aggregate for concrete was 1.95 mi., and the average haul for the bitulithic mixture was 1.35 mi. Crushed granite rock was furnished to the contractor from the county-owned quarry at Kapowsin

*County Engineer, Pierce County, Tacoma, Wash.

at cost, for use in the bitulithic top. The contractor paid freight on the rock at 1c. per ton per mile from Kapowsin to McMillan, a distance of 13 mi. Cement cost the contractor $2.30 per bbl. delivered on the work, with a rebate of 40c. per bbl. for sacks.

In the construction of this work a one-car asphalt plant was used by changing the sand bin into four compartments. The plant consists of a rotary drum to heat the sand and rock (oil being used for fuel), elevators to lift it into the elevated screens and separate it into the different sizes, and the bins into which the sand and rock pass after screening; there are three kettles, heated by coal, and staves of the barrels that contained bitulithic cement, in which the bitulithic cement is heated, a weighing machine having a scale of seven beams, and weighing platform, buckets and mixing boxes.

The crushed rock and sand are separately piled near the end of the rotary drum. A two-horse scraper was used to bring the rock and sand to the elevators, where they are shoveled into the elevators which empty into the heating drum. It takes about four minutes from the time the aggregate starts in the drum to reach the bin. The rock is heated from 240 to 250° F. After leaving the drum the aggregate is lifted onto the screens which separate it into four sizes, passing into as many bins.

No. 1 bin contains aggregate passing No. 10 mesh; No. 2 bin contains aggregate passing ¼-in. screen and retained on a No. 10; No. 3 bin passing ½-in. screen and retained on ¼-in.; No. 4 bin passing 1¼-in. square opening and retained on ½-in. screen.

The following proportions were generally used, though some little variation was made to fit different material: 315 lb. No. 1 bin; 90 lb. No. 2 bin; 140 lb. No. 3 bin; 440 lb. No. 4 bin; and 15 lb. of dust finer than 200 mesh. This made 1,000 lb. of aggregate, to which was added 88 lb. of bitulithic cement heated to 250° F.

These materials were carefully weighed, passed into a twin pug mill and thoroughly mixed for about 1 min. 40 sec., and then dumped into a wagon which stands underneath to receive it. It is hauled by teams and dumped onto the street, shoveled over until it is about the thickness required, raked and got as near a smooth surface as possible. It is then rolled with a 10-ton roller until thoroughly smooth and no appreciable compression appears and it does not creep under the roller. If any low places develop, a little more material is put on to bring it to the required grade and thickness. After it is rolled, a squeegee car is used to spread the bitulithic cement, which is heated in kettles on the street, and the top is thoroughly coated with the bitulithic cement, using approximately ¼-gal. per sq.yd., and then covered with hot screenings from No. 1 bin with a little excess sand and put on by a sand-spreading machine using approximately 25-lb. per sq.yd. The pavement is then thoroughly rolled with a 10-ton roller, which completes the process.

FIG. 2. ALDERTON-ORTING BITULITHIC ROAD, TACOMA, WASH.

The force required at the plant and the wages paid are as follows:

1 superintendent, furnished by the patentees	$200.00 per month
4 feeders, 3 using shovels and 1 a hoe, to feed the aggregate into the elevators	2.50 per day
1 foreman	100.00 per month
1 drum fireman	2.50 per day
1 boiler man	2.50 per day
1 hopper man	2.50 per day
1 mixer man	2.50 per day
1 team	5.00 per day
1 night watchman	3.00 per day

From 3 to 13 teams costing $5 per day were used to haul the material onto the pavement.

The street force was as follows:

1 foreman	$4.00 per day
1 roller man	3.50 per day
1 man, cleaning pavement	2.25 per day
1 wagon dumper	2.50 per day
1 tamper	2.50 per day
5 shovelers	2.50 per day
2 rakers	2.50 per day
2 squeegee men	2.50 per day

Development of Inland Ocean Port at Houston, Tex.

The entrance of Houston, Tex., into the ranks of the seaports of the United States dates from the completion of the Houston ship channel in the fall of 1914. The channel has an average depth of 26½ ft. from Bolivar Roads, opposite Galveston, to the turning basin within the city limits of Houston, a distance of 50 mi. This channel was excavated from 1912 to 1914 at a cost of $2,500,000, half of which was contributed by the County of Harris, which includes Houston, and half by the United States Government. Previous to that time an 18½-ft. channel had been excavated by the Government. A complete account of this work will be found in *Engineering News*, July 23, 1914, p. 188, in an article by Charles Crotty, United States Superintendent of Dredging.

The ship channel follows Buffalo Bayou, the use of which for light-draft boats dates back many years. The excavation of the channel has involved the dredging of 35,000,000 cu.yd. at an average cost of 6⅔c. per cubic yard. For maintaining and ultimately widening the channel the County of Harris also contributed $200,000 of the $400,000 required to build and equip two thoroughly uptodate 20-in. suction dredges. The United States agreed to pay for the maintenance and operation of the dredges.

The great storm of August, 1915, which caused so much damage in Galveston, shoaled the Houston ship channel in one or two places, so that its minimum width at the present time is 100 ft. at one place. The storm also sank one of the dredges, which, however, has been floated and is now undergoing the necessary repairs at the yards of the Bowers Southern Dredging Co., Galveston, builder of the two dredges. The port authorities of Houston hope ultimately to have a channel 250 ft. wide with a minimum depth of 35 ft.

Beginnings of the Port Work

The City of Houston took up the development of the municipal port soon after the completion of the ship channel. A bond issue of $3,000,000 was voted and sold. The construction of wharf and docking facilities was at once commenced under the supervision of E. E. Sands, City Engineer, who is ex-officio consulting engineer of the harbor commission. To Feb. 1, 1916, about $1,100,000 of the $3,000,000 had been expended, and contracts for the expenditure of about $1,000,000 were about to be let.

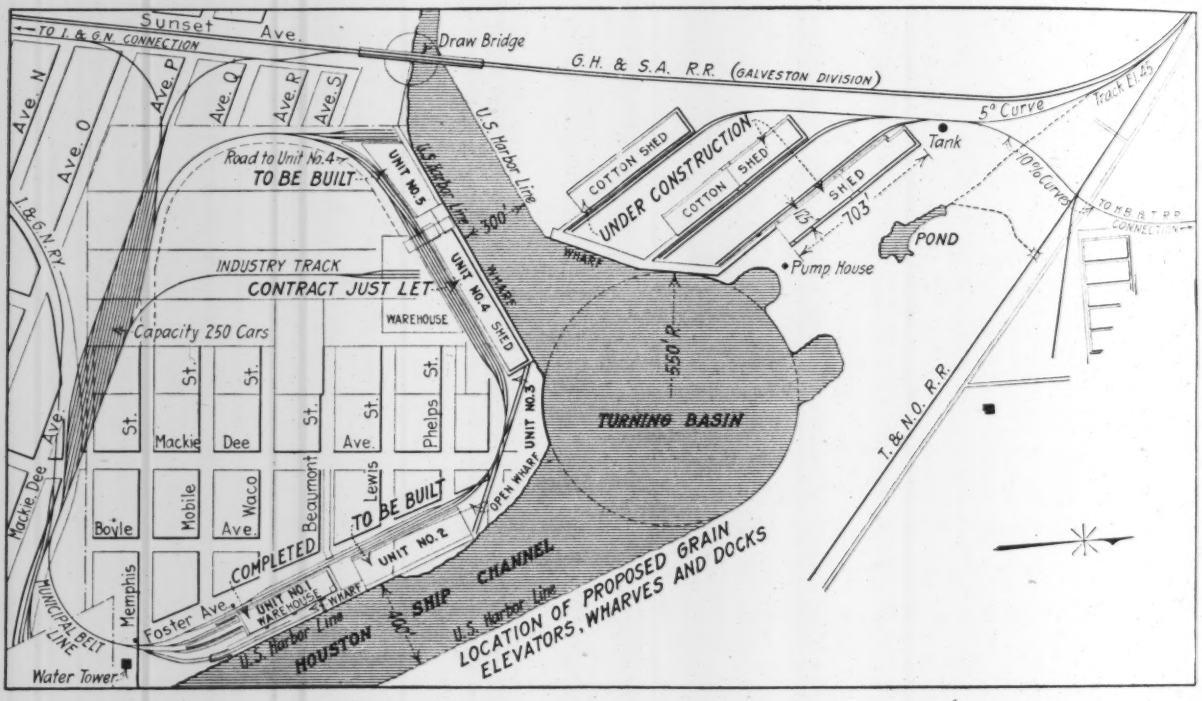


FIG. 1. GENERAL PLAN OF THE PORT OF HOUSTON, TEX., SHOWING COMPLETED AND PROJECTED DEVELOPMENTS

A general plan of completed and contemplated improvements is shown in Fig. 1. The work already finished includes shed and wharf No. 1. The wharf is a creosoted-pile and timber structure 625x50 ft. in plan and is located in front of a one-story concrete warehouse 425x100 ft. in plan and additional wooden sheds, rail connections and unloading platforms. This unit has cost $285,442. Fig. 3 is a view of it on the day it was put in service.

In constructing unit 1 all excavation above the level of the floor was considered and paid for as excavation. All excavation below the floor was considered in the contract as a part of the cost of the shed or wharf. The excavated material was used for filling, as directed by the city engineer. The concrete in the foundations was mixed 1:3:6; and in the walls and columns of the warehouse,

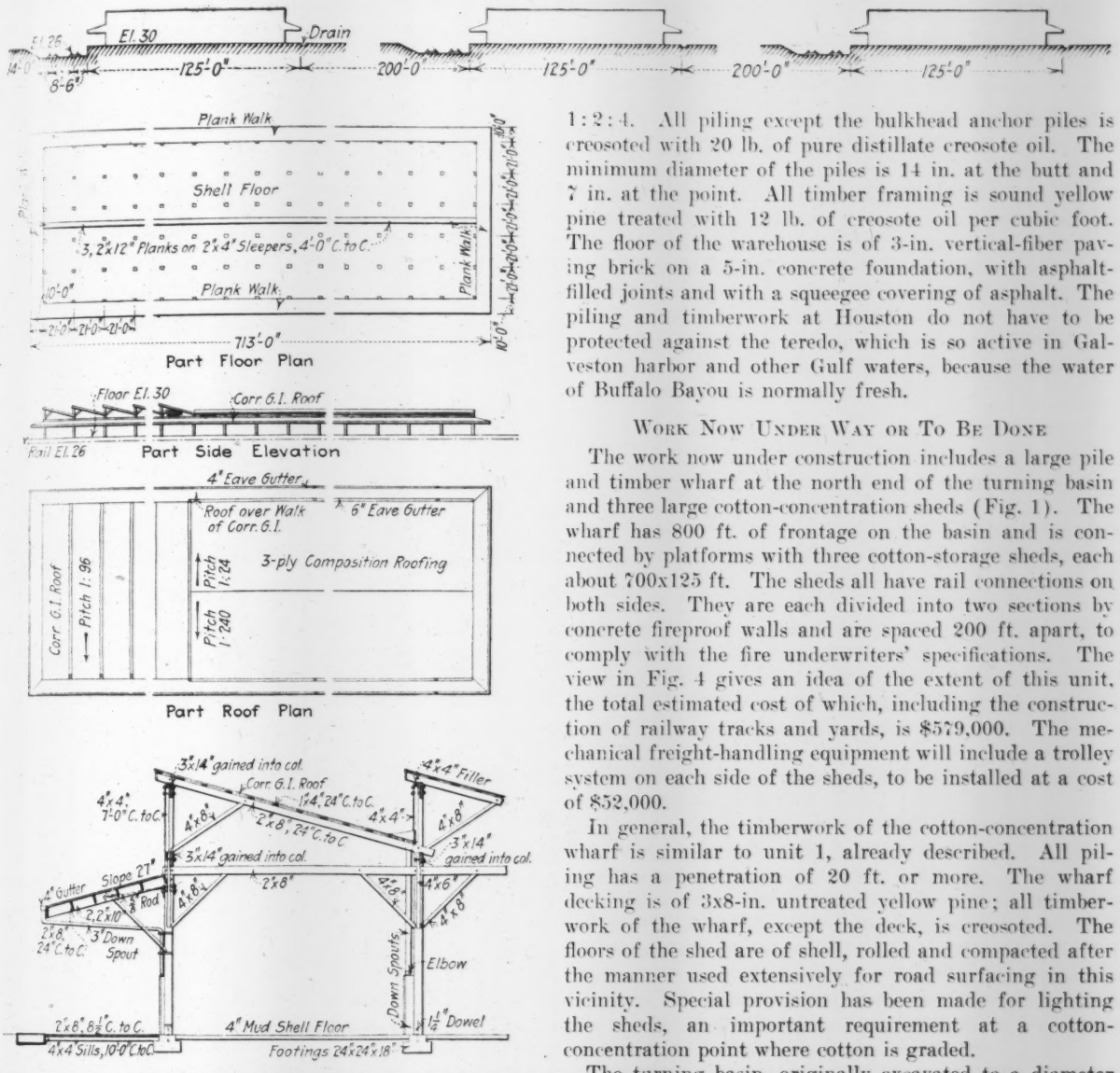


FIG. 2. DETAILS OF COTTON CONCENTRATION WHARF AND SHEDS; HOUSTON, TEX., PORT DEVELOPMENT

1:2:4. All piling except the bulkhead anchor piles is creosoted with 20 lb. of pure distillate creosote oil. The minimum diameter of the piles is 14 in. at the butt and 7 in. at the point. All timber framing is sound yellow pine treated with 12 lb. of creosote oil per cubic foot. The floor of the warehouse is of 3-in. vertical-fiber paving brick on a 5-in. concrete foundation, with asphalt-filled joints and with a squeegee covering of asphalt. The piling and timberwork at Houston do not have to be protected against the teredo, which is so active in Galveston harbor and other Gulf waters, because the water of Buffalo Bayou is normally fresh.

Work Now Under Way or To Be Done

The work now under construction includes a large pile and timber wharf at the north end of the turning basin and three large cotton-concentration sheds (Fig. 1). The wharf has 800 ft. of frontage on the basin and is connected by platforms with three cotton-storage sheds, each about 700x125 ft. The sheds all have rail connections on both sides. They are each divided into two sections by concrete fireproof walls and are spaced 200 ft. apart, to comply with the fire underwriters' specifications. The view in Fig. 4 gives an idea of the extent of this unit, the total estimated cost of which, including the construction of railway tracks and yards, is $579,000. The mechanical freight-handling equipment will include a trolley system on each side of the sheds, to be installed at a cost of $52,000.

In general, the timberwork of the cotton-concentration wharf is similar to unit 1, already described. All piling has a penetration of 20 ft. or more. The wharf decking is of 3x8-in. untreated yellow pine; all timberwork of the wharf, except the deck, is creosoted. The floors of the shed are of shell, rolled and compacted after the manner used extensively for road surfacing in this vicinity. Special provision has been made for lighting the sheds, an important requirement at a cotton-concentration point where cotton is graded.

The turning basin, originally excavated to a diameter of 600 ft., is being widened to a diameter of 1,100 ft.

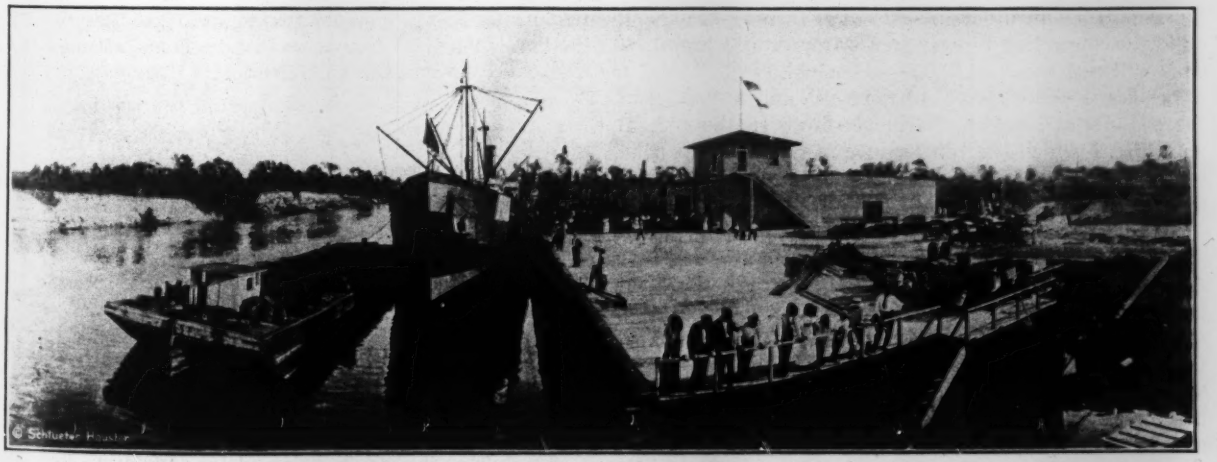


FIG. 3. UNIT NO. 1 IN THE MUNICIPAL DEVELOPMENT OF THE PORT OF HOUSTON ON THE OPENING DAY, SEPT. 26, 1915

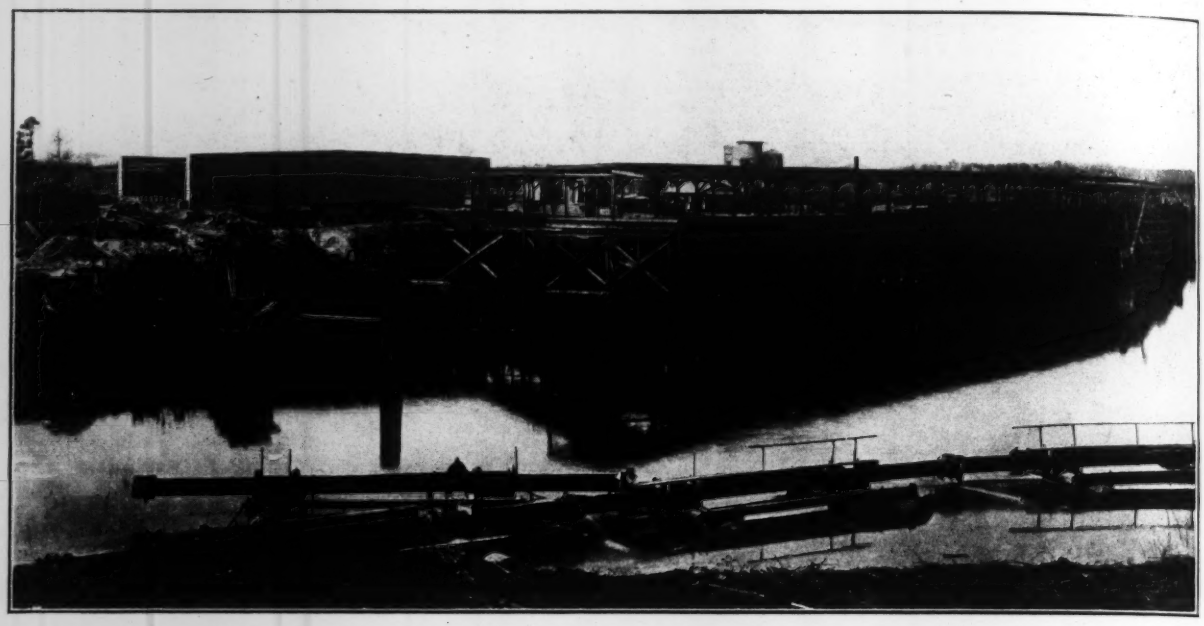


FIG. 4. COTTON-CONCENTRATION WHARF AND END OF ONE SHED, SHOWING CONCRETE FIREPROOF WALLS

This work is under contract with the Atlantic, Gulf & Pacific Co., working with the 20-in. suction dredge "Texas." This dredge, while working in the channel below Houston, made the remarkable record of 1,260,000 cu.yd. in one month. An idea of the amount of excavation required for this basin may be had when it is realized that the banks of the basin are some 20 to 30 ft. above the water level. The city already owns 237 acres of land fronting the basin and channel, and the purchase of additional land is contemplated.

The work for which contracts have just been let, as announced last week, is designated on the general plan as unit No. 4. It consists of a 777x20-ft. concrete wharf (resting on timber piles cut off below water surface) and a reinforced-concrete shed 660x100 ft., with a large warehouse in the rear. The construction of the belt-line railway to serve units 1, 2, 3 and 4 is included in this development, together with a railway-car storage yard with a capacity to handle 500 cars. The estimated cost of this unit is $579,000, not including the excavation for wharves Nos. 2 and 3, which will be necessarily done at the same time in order to widen the turning basin and build the connecting railway tracks from unit 1 to unit 4. This will cost about $112,000 additional.

Besides the development of the south and west sides of the channel and turning basin as shown in the general plan, Fig. 1, which is all included under the original bond issue, it is proposed to build at once, opposite unit 1, the first section of a large grain elevator, workhouse, track sheds, driers and loading-conveyor galleries at a cost of $731,000. The proposed unit of the grain elevator will have an initial capacity of 780,000 bu.; and the workhouse, 230,000 bu. These structures will stand some distance from, and higher than, the channel and will be connected with the wharf by housed belt conveyors. The ultimate development on this side of the channel, for which the city has already purchased the land, includes three wharves and warehouses and a railway yard for 2,500 cars. To construct one unit and a yard for 800 cars will require $920,000. This part will be built first.

All the developments thus far described will be owned and operated by the City of Houston. The belt-line railway and all the railway yards and wharf connections will be operated as a municipal enterprise. It is proposed to establish zones of uniform charges for transportation from all points within the city. The municipal railway has connections with all the city railway systems, which, counting as distinct the separate division of the same railway, number seventeen. Houston is already one of the largest concentration and shipping points for cotton. The cotton shipments already amount to approximately 3,000,000 bales annually.

Besides the municipal harbor development a private enterprise has on foot a scheme for developing the channel of Buffalo Bayou above the turning basin and nearer the city at a total estimated cost of about $10,000,000.

Houston business men have become very optimistic concerning the opportunities of Houston as a seaport; and if all the schemes now projected are carried out, Houston will doubtless become the principal Gulf port. A very essential part of the program, however, will be the widening of the 50-mi. channel to the sea. Needless to say, the development of Houston as a port is not looked upon with much enthusiasm in Galveston.

Texas Sewage-Works Activity

More sewage-works are likely to be constructed in Texas in 1916 than in any five years heretofore, according to V. M. Ehlers, State Sanitary Engineer, Austin, Tex. Much of this activity is due to the law for preventing the pollution of watercourses, passed by the state legislature in 1913 and amended in 1915. This law virtually prohibits the discharge of untreated sewage into any stream or other water of the state, except tidal water, after Jan. 1, 1917. Even tidal waters were included in the prohibition until the amendment of 1915. The body of the law is as follows:

Sec. 1. That it shall be unlawful for any person, firm or corporation, private or municipal, to pollute any watercourse or other public body of water from which water is taken for

the uses of farm, live-stock, drinking and domestic purposes, in the State of Texas, by the discharge, directly or indirectly, of any sewage or unclean water or unclean or polluting matter or thing therein, or in such proximity thereto as that it will probably reach and pollute the waters of such watercourse or other public body of water from which water is taken, for the uses of farm, live-stock, drinking and domestic purposes; provided, however, that the provisions of this bill shall not affect any municipal corporation situated on tidewater; that is to say, where the tide ebbs and flows in such watercourse. A violation of this provision shall be punished by a fine of not less than $100 and not more than $1,000. When the offense shall have been committed by a firm, partnership or association, each member thereof who has knowledge of the commission of such offense shall be held guilty. When committed by a private corporation, the officers and members of the board of directors having knowledge of the commission of such offense shall each be deemed guilty; and when by a municipal corporation, the mayor and each member of the board of aldermen or commission having knowledge of the commission of such offense, as the case may be, shall be held guilty as representatives of the municipality; and each person so indicated as above shall be subject to the punishment provided hereinbefore; provided, however, that the payment of the fine by one of the persons so named shall be a satisfaction of the penalty as against his associates for the offenses for which he may have been convicted; provided, the provisions of this act shall not apply to any place or premises located without the limits of an incorporated town or city, nor to manufacturing plants whose affluents contain no organic matter that will putrefy, or any poisonous compounds, or any bacteria dangerous to public health or destructive of the fish life of streams or other public bodies of water.

Sec. 2. Upon the conviction of any person under Sec. 1 of this act, it shall be the duty of the court, or judge of the court, in which such conviction is had, to issue a writ of injunction, enjoining and restraining the person or persons or corporation responsible for such pollution, from a further continuance of such pollution; and for a violation of such injunction, the said court and the judge thereof shall have the power of fine and imprisonment, as for contempt of court, within the limits prescribed by law in other cases; provided, that this remedy by injunction and punishment for violation thereof shall be cumulative of the penalty fixed by Sec. 1 of this act; and the assessment of a fine for contempt shall be no bar to a prosecution under Sec. 1; neither shall a conviction and payment of fine under Sec. 1 be a bar to contempt proceedings under this section.

Sec. 3. Any city or town of this state, with a population of more than 50,000 inhabitants, which has already an established sewerage system dependent upon any watercourse or other public body of water, from which water is taken for the uses of farm, live-stock, drinking and domestic purposes, or which discharges into any watercourse or public body of water, from which water is taken for the uses of farm, live-stock, drinking and domestic purposes, shall have until Jan. 1, 1917, within which to make other provisions for such sewage. Cities and towns of less population than 50,000 inhabitants shall have until Jan. 1, 1917, within which to make other arrangements for the disposal of such sewage. Any person, firm or corporation, private or municipal, coming under or affected by the terms of this bill, or any independent contractor having the disposal of the sewage of any city or town, shall have until Jan. 1, 1917, within which to make other arrangements for the disposal of such sewage, or other matter which may pollute the water, as defined in this bill.

Sec. 4. The Texas State Board of Health is authorized, and it is hereby made its duty, to enforce the provisions of this act; and to this end the Governor shall appoint, by and with the consent of the Senate, an inspector to act under the direction of the said Board of Health and the state health officer making such investigations, inspections and reports, and performing such other duties in respect to the enforcement of this act as the said Board of Health officer may require.

It is claimed that there are some loopholes in the law, and its constitutionality is being freely questioned. It makes no provision for financing the work, and under the laws of the state governing the issue and sale of municipal bonds it has frequently been practically impossible to inaugurate much needed public improvement. It would seem doubtful under the circumstances if a mayor or city commissioner could be held responsible if the public would not or could not finance the work. The law specifically states that the issuing of the injunction must succeed the conviction of the officers, so if it turns out to be impossible to convict the officers the law would seem to be of no effect.

Notwithstanding the doubtful character of the law there is evidence that nearly all the municipalities of the state are willing to comply with its spirit. To comply with the letter of it would of course entail the purification of sewage to the degree required for a water-supply. Such an interpretation will not be made by the state officials. Mr. Ehlers says:

> In enforcing the stream-pollution act the economic aspect of the case will be considered; namely, the results accomplished must be reasonably commensurate with the cost of prevention of pollution. In other words, the State Board of Health places a liberal interpretation upon the word pollution in the act. Our work up to the present time has been more or less along educational lines. Where a water-supply becomes suspicious, we advise treating the water.

Mr. Ehlers has issued a preliminary report on "Standards of Purity of Sewage Effluents," in which is stated as the object "to arrive at some temporary standard or working basis." After defining pollution and the tests by which pollution is to be determined, the report closes with the following summary:

> Whereas, it is inevitable that absolute prevention of pollution is impossible, that this state has no control over the Canadian, Pecos, Rio Grande and Red Rivers outside of the state boundary, and recognizing the process of self-purification of streams, it would be needless expense to require all municipalities to purify their sewage to a drinking-water standard. It is recommended that with minor changes the standards from the report of the Committee on Standards of Purity . . . [of the National Association for Preventing the Pollution of Rivers and Waterways, see "Engineering News," Oct. 31, 1913] be adopted as a working basis for the future.

Of the four largest cities in the state, San Antonio disposes of its sewage by irrigation, Dallas has under construction an Imhoff-tank disposal plant, Houston is getting ready to install an activated-sludge plant and Fort Worth has taken no definite steps toward the construction of a plant. Of the 130 cities and towns in Texas having sewer systems, all but two, Mr. Ehlers states, have shown an inclination to build disposal works. A score or more already have works under way. The capital city of Austin has had surveys and recommendations made, but no definite steps have been taken. The City of Waco, which discharges its raw sewage into the Brazos River, has shown a disposition to contest the law.

The consulting service of the State Sanitary Engineer is free to any municipality requesting it. During his first three months of office specific information was given by him in answer to about fifty requests of this kind. All designs for sewerage systems and dispoal works must be passed upon by him.

A great deal of the sewer-designing work of Texas municipalities has been and is at present being done by firms of consulting engineers, both in and outside the state. Competition for much of this work is very keen and appears not to be governed by any existing code of engineering ethics. One effect of the new law will be to protect the towns and cities from the designs of pseudo and incompetent engineers, as has been already accomplished in Kansas under a similar but less drastic law.

There seems to be no doubt that the law will serve—in fact, already has served—a useful purpose. Mr. Ehlers states that while the law is far from being a model, "it is gratifying to know that it has caused an awakening of the public toward better sanitation; municipalities are not only submitting plans for sewage-treatment works, but are anxious for advice on sanitation problems."

Diagram for Full Comparison of Hydraulic Turbines

By Robert E. Horton* and Walter C. Pomeroy†

SYNOPSIS—A new form of efficiency-contour diagram is developed, using proportionate speeds and powers, referred to speed and power at maximum efficiency, instead of absolute quantities, as has been customary. When unity speed is taken as the "type characteristic" of a runner under study (speed at which a model runner would develop 1 hp. under 1-ft. head), the diagram approximately applies to all sizes of runners having identical features.

With a view to devising a graphical means whereby the suitability of different hydraulic turbine runners to meet given requirements can be deduced, the writers have developed the type of efficiency diagram presented in the following paragraphs and accompanying curves. Somewhat similar methods are in general use now, although

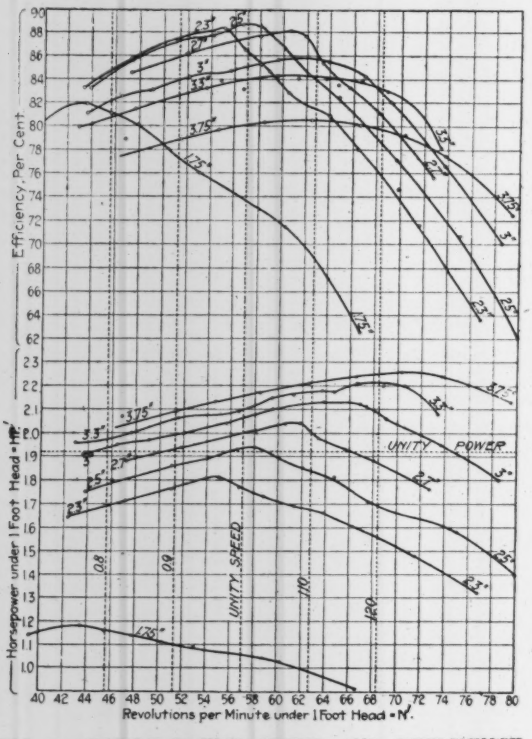


FIG. 1. SIMPLE POWER-SPEED AND EFFICIENCY-SPEED CURVES

For 23-in. right-hand hydraulic turbine runner built by Hydraulic Turbine Corporation; Holyoke test 2,426, Dec. 1, 1915.

it must be said that so far no single one has been generally accepted. In the type of diagram here presented the ratios of various speeds and powers to the speed and power for maximum efficiency are utilized instead of absolute quantities. The diagrams have a further usefulness; by making the unity power 1 hp. under 1-ft. head the unity speed corresponds to the "type characteristic" of the runner at its best efficiency points.

This type of diagram shows the efficiency at which a turbine could operate when running at various proportionate speeds and when producing various percentages of unity power (unity power being capacity at maximum efficiency). The diagram will be made to show at what proportionate speed the turbine would have to operate

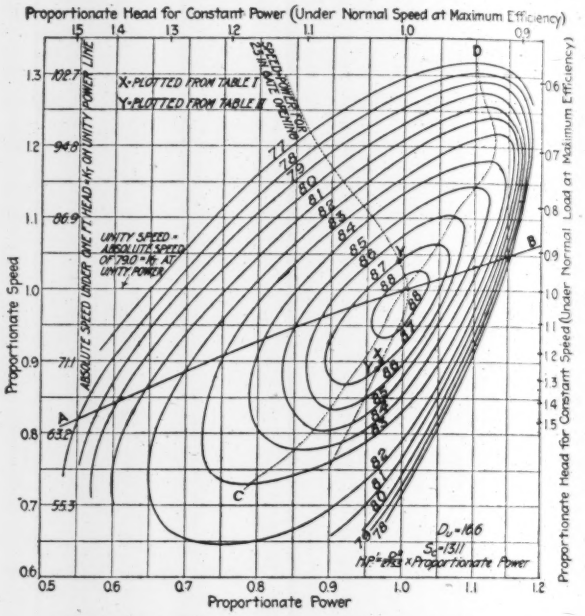

Proportionate Head for Constant Power (Under Normal Speed at Maximum Efficiency)
Proportionate Speed
Proportionate Power
Proportionate Head for Constant Speed (Under Normal Load at Maximum Efficiency)
X-PLOTTED FROM TABLE I
Y-PLOTTED FROM TABLE II
UNITY SPEED = ABSOLUTE SPEED OF 790 = N AT UNITY POWER
ABSOLUTE SPEED UNDER ONE FT HEAD = N' OF UNITY POWER LINE
SECONDARY FOR 25% GATE OPENING
A
B
C
D

FIG. 2. EFFICIENCY-SURFACE DIAGRAM FOR PROPORTIONATE SPEEDS AND POWERS

Showing one of the typical speed-power-gate curves used in developing the constant-efficiency contours. For 23-in. runner of Holyoke test 2,426.

to maintain a constant speed under different heads and also to show the efficiency which would be developed for any given actual power output under the various heads. Moreover, it will serve as an absolute and final comparison between runners of different type characteristics, and by the introduction of proper constants any possible combination of power, speed and diameter suiting a particular runner design can be worked out. These uses will be shown by examples.

Development of Diagram

The development of the new type of efficiency surface diagram is illustrated by employing Holyoke test data from a 23-in. and a 30-in. runner built by the Hydraulic Turbine Corporation. First of all, the ordinary power-speed and efficiency-speed curves are plotted by reducing the Holyoke data to a constant head of 1 ft. These curves are shown in Fig. 1. On this chart will be seen the unity-speed and unity-power lines (which pass through the point of maximum efficiency) and a few of the proportionate-speed lines that form the basis of the final diagram.

Reading along the proportionate-speed line 0.90 (absolute speed $N' = 51.3$), the absolute power and effi-

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ciency at the various gate openings may be tabulated, as shown in Table 1. In the fourth column of this table is the ratio absolute power (column 2) to power at point of maximum efficiency. Such a table is made for each proportionate-speed line. Plotting the values in column 4 of Table 1 (for line 0.90) on the abscissa for a proportionate speed of 0.90 in Fig. 2, we arrive at one point on the speed-power curves for each gate opening; the other tables, not here given, supply the remaining points for the curves, one of which is shown complete (for 2.5-in. gate opening).

Next, the values of efficiency in column 3, Table 1, are plotted against proportionate speed, as shown by Fig. 3 (along line 0.90 for this table), giving a speed-efficiency curve for each gate opening. Tabulating the values of the proportionate-speed points at which each constant-efficiency ordinate cuts the various gate-opening curves of Fig. 3, we secure Table 2 (though this table is really unnecessary in practice) for each efficiency contour desired on Fig. 2; and from Table 2 we determine a constant-efficiency point on each speed-power-gate curve on Fig. 2, these points determining the constant-efficiency surface.

It is readily seen that Fig. 2 is plotted in such a manner as to show the efficiency at which a turbine could operate when running at various relative speeds, expressed in terms of the speed of the maximum efficiency as unity, and when producing different amounts of power, expressed in terms of the horsepower at maximum efficiency as unity. For example, if owing to backwater conditions, the head is reduced so that in order to operate at constant speed it is necessary to run the turbine at a speed 10% greater than the speed of greatest efficiency, then the diagram shows that, when operating at normal power under the reduced head, the efficiency would be about 85%.

TABLE 1. ABSOLUTE HORSEPOWER UNDER 1-FT. HEAD, EFFICIENCY AND PROPORTIONAL HORSEPOWER; AT PROPORTIONATE SPEED 0.9 OF 23-IN. RUNNER, HOLYOKE TEST 2,426

Gate Opening, In.	Absolute Hp.*	Efficiency	Proportionate, Hp.
1.75	1.1	77.2	0.573
2.3	1.77	87.6	0.921
2.5	1.86	87.3	0.969*
2.7	1.93	86.0	1.005
3.0	1.99	84.0	1.036
3.3	2.06	82.5	1.072
3.75	2.09	79.0	1.09

* See Fig. 2.

TABLE 2. PROPORTIONATE SPEED POINTS; AT WHICH THE EFFICIENCY LINE 87% CUT THE GATE-OPENING CURVES IN FIG. 3

Gate Opening, In.	Prop. Speed	Prop. Speed
1.75	.....	.....
2.3	0.870	0.970
2.5	0.887	1.042*
2.7	0.955	1.065
3.0	.....	.....
3.3	.....	.....
3.75	.....	.....

* See Fig. 2.

TABLE 3. PROPORTIONATE SPEED AND POWER FOR CONSTANT SPEED OR CONSTANT POWER FROM A HYDRAULIC-TURBINE RUNNER OPERATING UNDER VARYING HEAD

Ratio Actual to Normal Head, $\frac{H_2}{H_1}$	Proportionate Speed for Constant Speed	Proportionate Power for Constant Power
0.50	1.4242	2.828
0.60	1.2923	2.151
0.70	1.196	1.707
0.80	1.118	1.398
0.90	1.054	1.171
1.00	1.0	1.0
1.10	0.9534	0.8665
1.20	0.91285	0.7610
1.30	0.8769	0.6748
1.40	0.84498	0.6039
1.50	0.81670	0.5444

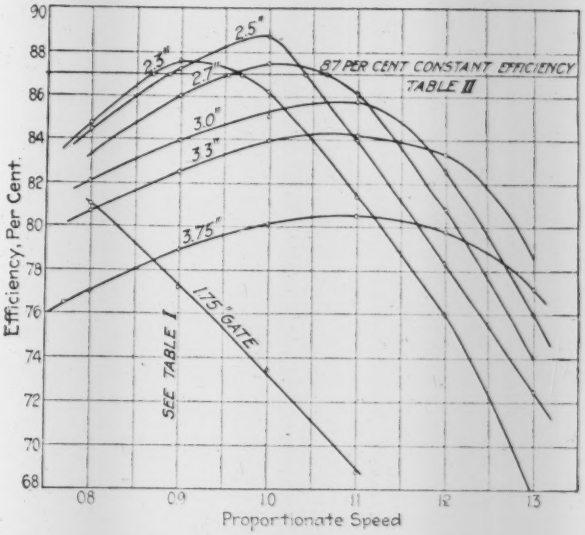

FIG. 3. PROPORTIONATE SPEED AND EFFICIENCY CURVES ACCOMPANYING FIGS. 1 AND 2

The diagram does not show directly, however, at what proportionate speed the turbine would have to run to maintain constant speed under different heads, nor does it show directly what the efficiency would be for any given actual power output under different heads. In order to determine these data from such a diagram in the simplest manner possible, two additional scales may be added to the figure—one for head corresponding to different relative speeds, the other giving the power corresponding to different relative heads. These scales are derived in the following manner:

Since the speed of a turbine operating at a given efficiency under any head is proportional to the square root of the head, it is evident that, if a turbine is set to operate at some particular speed, say, for example, its speed of maximum efficiency under a stated head, this speed being taken as unity, then in order to maintain this speed at either a greater or a less head it must under the changed conditions operate at a speed less or greater than the normal speed for the changed head and inversely proportional to the square root of the ratio of the original head to the changed head. Similarly, since the horsepower of a turbine varies as the three-halves power of the head, it is evident that to produce constant power out-

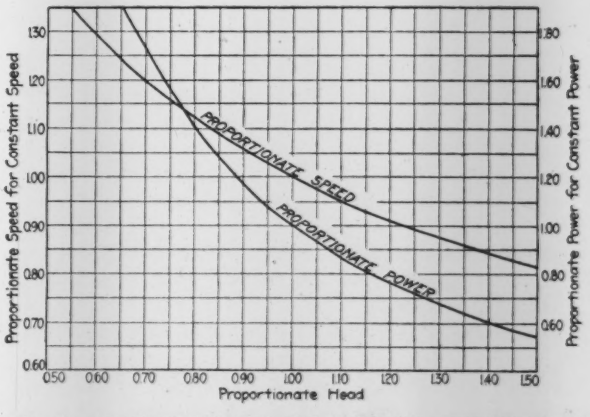

FIG. 4. PROPORTIONATE SPEEDS AND POWERS FOR CONSTANT SPEED OF POWER UNDER VARYING HEADS

put under different heads the power (constant) under the changed head must be inversely proportional to the three-halves power of the ratio of original to changed head.

The ratios, showing the proportionate speed at which a turbine must run in order to operate at constant speed under varying head, are shown in column 2 of Table 3 and graphically in Fig. 4, which is useful for finding intermediate points. Similarly, the proportionate power that the turbine must produce in order to produce a constant power output under different heads is shown in column 3 of the table and in Fig. 4, column 1 being in each case the ratio of the changed head to the standard or original head, which is taken as unity. Plotting the values given in columns 1 and 2 of Table 3 alongside of the scale of proportionate speeds on Fig. 2, the scale showing proportionate heads is obtained. Similarly, plotting the data from columns 1 and 3 alongside of the scale of proportionate power, the scale showing proportionate head for constant power is obtained. It is now possible with the aid of these additional scales to read from the efficiency-surface diagram directly the efficiency and power that can be obtained when the turbine is under varying heads, as under backwater conditions.

Since the speed varies as the square root and the power varies as the three-halves power of the head, it is evident that a certain change in head produces different changes in the relative speed and relative power respectively. Since the efficiency is constant, as a rule, only when the turbine is operating at a certain relative speed and power, it is evident that, in general, constant efficiency cannot be obtained when a turbine operates under a varying head. The question arises, What efficiency can be obtained from a turbine operating at constant speed and producing constant power output under varying head?

This is easily determined from the diagram. For this purpose plot a line AB on the diagram running through points corresponding to the same proportionate heads on the speed and power scales respectively. This line will pass through the point chosen as the point of normal power and speed—that is, the point O, or point of maximum efficiency in this case. The line AB intersects the contours of efficiency at points corresponding to certain proportionate heads for constant power and constant speed, and the efficiencies at these points of intersection are those which could be obtained with a turbine operating at constant power and speed under the corresponding relative heads.

Use in Selection of Turbines

If it is required that the turbine shall operate at constant speed under varying heads, but if a constant power output is not required, higher efficiencies may be obtained than under conditions of both constant power and constant speed. The line CD on the diagram is drawn through the maximum efficiencies that could be obtained with a turbine operating at constant speed under the proportionate heads indicated on the scale of speeds at the right of the diagram for normal speed at the point of maximum efficiency.

It is generally true that, having given a turbine of a certain size, which has a certain K_t, or, in other words, a certain maximum absolute efficiency and certain speed and power under 1-ft. head, another turbine of larger or smaller diameter of identical features and proportionate dimensions will have substantially the same efficiency and K_t, so that with a diagram of this form the availability of a turbine of the right size to meet certain conditions can be determined with regard to those conditions from a test of a similar turbine of larger or smaller size. (At least this method seems to give the best working hypothesis in cases where a turbine of the actual size required for a particular case has not been tested. It is well known of course that the efficiency rises slightly as the size of the turbine is increased.)

The power and speed under 1-ft. head at best efficiency of the 23-in. runner used for illustration here have been shown on Fig. 1 to be $HP.' = 1.92$ $N' = 57$ respectively. As already stated, the type characteristic is independent of the size, remaining a constant for the runner series. By definition it is the speed at which a turbine would run under 1-ft. head if it was so reduced or enlarged in all its proportions as to give 1 hp. under 1-ft. head, or expressed numerically,

$$K_t = N' \sqrt{HP.'}$$

Thus, from the foregoing,

$$K_t = 57 \sqrt{1.92} = 79$$

Reduce the dimensions of the runner under study so that the absolute value of unity power on Fig. 2 actually will be 1 hp. under 1-ft. head instead of 1.92. Since K_t remains a constant; therefore, $79 = N' \sqrt{1}$, or the best speed (unity on Fig. 2) under 1-ft. head of the hypothetical wheel will be 79 instead of 57. As a means of comparison between runner types this diagram now shows at a glance the absolute criterion—the K_t. By its use we are comparing runners all capable of exactly the same amount of work—1 hp. under 1-ft. head. Unity power will be the same for all, and unity speed will distinguish the curve form just as readily and fully as K_t does the runner type.

It must be further noticed that the comparison is between the speed of runners, all capable of developing the same power, rather than between the power of runners at the same speed or of the same diameter. The latter comparisons appeal to our imagination, but in reality are not the logical view; in fact, a comparison based on runners of the same diameter is technically wrong.

Now, with the addition of two constants we have the basis for all ordinary computations of horsepower, speed and diameter, suiting the particular runner design. We have introduced what we are pleased to call a "unit diameter," D_u and a "speed constant," S_c. The D_u is nothing more or less than the diameter in inches of our hypothetical runner, which develops 1 hp. under 1-ft. head. The S_c is the product of D_u and K_t. By introducing these constants in the following equations we can solve for the required unknowns:

$$D = \sqrt{\frac{HP.}{H^{\frac{3}{2}}}} D_u = \frac{S_c \sqrt{H}}{R.P.M.}$$

$$R.P.M. = \frac{S_c \sqrt{H}}{D}$$

in which D = diameter, in inches, of a runner that will give $HP.$ (horsepower) at *best efficiency* under H (feet head) at $R.P.M.$ (revolutions per minute).

Before going farther into the use of these diagrams it may be wise to review the subject of "type character-

istic" or "unit speed" in the briefest possible manner. It is self-evident that with almost every conceivable combination of horsepower, head and speed to meet, there must necessarily be some very radically different designs of turbine runners. It is just as imperative that we have a ready and sure means of comparison between the many designs; also that, with given conditions at a plant, we may have some means by which we can immediately eliminate the great number of runner designs not at all suited and centralize our study on just those designs fitting our problem. Such a means of comparison between runner types and of connection between local conditions and runner designs must necessarily be based on the work which the runner can perform; that is, we must have some simple expression of runner duty which will stand out distinctly as a special characteristic of different runners and may also be expressed in terms of local plant conditions. The so-called "type characteristic" or "unit speed" is just the desired expression.

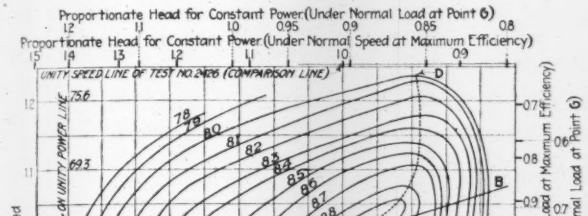
Proportionate Head for Constant Power (Under Normal Load at Point 6)
Proportionate Head for Constant Power (Under Normal Speed at Maximum Efficiency)
UNITY SPEED LINE OF TEST NO. 2426 (COMPARISON LINE)
UNITY POWER LINE
ABSOLUTE SPEED UNDER UNIT HEAD = N' ON UNITY POWER LINE
Q = UNITY SPEED = ABSOLUTE SPEED OF .83 EFF. AT UNITY POWER
Proportionate Speed
Proportionate Power
Relative Head for Constant Speed (Under Normal Load at Maximum Efficiency)
Relative Head for Constant Speed (Under Normal Load at Point 6)
Normal HP. (under one foot head) = D² (New Size) = D²/441
Sc = 1323
Dc = 21

FIG. 5. EFFICIENCY-SURFACE DIAGRAM FOR NEW MILL-TYPE TURBINE

For 30-in. right-hand runner built by Hydraulic Turbine Corporation; Holyoke test 2,267

We neglect all design details entirely, reduce all runners to the same capacity and then compare their speeds. Such a classification is based definitely on capacity and speed. The type characteristic (K_t) is the speed under 1-ft. head of a turbine runner so reduced or enlarged in all its proportions as to give 1 hp. under 1-ft. head. Expressed numerically, it is

$$K_t = \frac{R.P.M. \sqrt{HP.}}{H^{\frac{5}{4}}}$$

which is also an expression of the three controlling factors at any site—the horsepower, the revolutions per minute and the head in feet.

Revolutions per minute under 1-ft. head $= N' = \frac{R.P.M.}{H^{\frac{1}{2}}}$; horsepower under 1-ft. head $= HP.' = \frac{HP.}{H^{\frac{5}{2}}}$; and

$$K_t = \frac{R.P.M. \sqrt{HP.}}{H^{\frac{5}{4}}} = N' \sqrt{HP.'}$$

Let $HP.' = 1$; then $K_t = N'$. The K_t of a multiple runner unit is greater than that of a single runner in the ratio of the square root of the number of runners in the unit. To facilitate the calculation of K_t a table of five-fourths powers of numbers is presented herewith as Table No. 6.

Examples of Turbine Study

The uses of the diagrams of Fig. 2 may be illustrated by two examples. For the first, assume a normal head H of 20 ft. and a constant speed $R.P.M.$ of 117. A maximum capacity of 900 hp. is desired under 20-ft. head at not less than 83% efficiency. Now

$$K_t\ (maximum) = \frac{117 \sqrt{900}}{20^{\frac{5}{4}}} = 83$$

We can readily see from the diagrams that the type of runner is closely shown by test 2,426 for the normal $K_t = 79$. On Fig. 2 read along unity-speed line beyond unity power until it crosses about 83% efficiency line, which is at 1.1 proportionate power. This represents maximum power desired. Therefore, the normal power of the runner will be $90 \div 1.1 = 815$. Then

$$D = \sqrt[2]{\frac{900}{20^{\frac{5}{2}}}} \times 16.6 = 50\ in.$$

Unity power of the 50-in. runner $= \frac{50^2}{275.5} = 9.1$; and the efficiency and power at the various heads are shown in Table 4, values being found along the line CD, Fig. 2.

For the second problem assume given a normal head of 25 ft., and required a maximum of 300 hp. under normal head and at a speed of 220 r.p.m.

$$Maximum\ K_t = \frac{220 \sqrt{300}}{25^{\frac{5}{4}}} = 68$$

This value leads us to try the runner of test 2,267, which has a normal K_t of 63.

$$D = \frac{S_c \times \sqrt{H}}{R.P.M.} = \frac{1{,}323 \times 5}{220} = 30\ in.$$

The normal $HP.'$ for a 30-in. runner is $30^2 \div 441 = 2.04$ (see Fig. 5), and $2.04 \times 25^{\frac{5}{2}} = 255$ hp. normal. The required maximum power corresponds to $300 \div 225 = 1.18$ proportionate power. The proportionate power of 1.18 on the normal-speed line gives 82% efficiency—which is a good value for full gate.

The efficiency and horsepower at the various heads are shown in Table 5 under column 3 and column 6 respectively. The values of efficiency and proportionate horse-

TABLE 4. EFFICIENCY AND HORSEPOWER; OBTAINABLE UNDER VARYING HEAD, CONSTANT SPEED FOR 50-IN. RUNNER OF TEST 2,426

Head, Ft.	Proportionate Head	Efficiency at Constant Speed	Proportionate Hp. at Constant Speed	Hp.'	Actual B.Hp.
18	0.9	87.2	1.042	9.5	725
19	0.95	88.0	1.022	9.3	770
20	1.0	88.8	1.0	9.1	815
21	1.05	88.7	0.99	9.0	875
22	1.1	88.3	0.975	8.86	915
23	1.15	88.0	0.965	8.77	970
24	1.2	87.6	0.952	8.66	1,020

TABLE 5. POWER AND EFFICIENCY OF 30-IN. WHEEL OF TEST 2,267; ACCORDING TO PROBLEM 2

(1) Head, Ft.	(2) Proportionate Head	(3) Efficiency at Constant Speed, 220 R.p.m.	(4) Proportionate Hp. for Constant Speed, 220 R.p.m.	(5) Absolute Hp.'	(6) B.Hp.	(7) Efficiency for Constant R.p.m. = 220—Hp. = 255, Point O	(8) Efficiency for Constant R.p.m. = 185—Hp. = 215, Point G
20	0.80	86.2	1.11	2.26	202	....	*76.0
21	0.84	87.3	1.105	2.25	216	....	84.1
22	0.88	88.2	1.092	2.23	230	80.0	86.5
23	0.92	88.6	1.074	2.19	240	86.0	87.0
24	0.96	88.7	1.04	2.12	249	88.5	87.0
25	1.00	88.9	1.0	2.04	255	88.9	84.8
26	1.04	88.7	0.975	2.00	265	88.3	83.0
27	1.08	88.5	0.955	1.96	275	87.0	81.0
28	1.12	88.4	0.94	1.925	285	86.2	79.0
29	1.16	88.2	0.93	1.9	297	85.0	....

TABLE 6. VALUES OF $H^{\frac{3}{2}}$ FOR CALCULATING TYPE CHARACTERISTIC (ROBERT E. HORTON)

H	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
4	5.66	5.83	6.01	6.19	6.37	6.55	6.74	6.92	7.10	7.29
5	7.48	7.66	7.85	8.04	8.23	8.42	8.62	8.81	9.00	9.20
6	9.39	9.59	9.78	9.98	10.18	10.38	10.58	10.78	10.98	11.18
7	11.39	11.59	11.79	12.00	12.21	12.41	12.62	12.83	13.04	13.24
8	13.45	13.67	13.88	14.09	14.30	14.51	14.73	14.94	15.16	15.37
9	15.59	15.81	16.02	16.24	16.46	16.68	16.90	17.12	17.34	17.56
10	17.78	18.00	18.23	18.46	18.68	18.90	19.13	19.35	19.58	19.80
11	20.03	20.26	20.49	20.72	20.95	21.18	21.41	21.64	21.88	22.11
12	22.34	22.57	22.80	23.04	23.27	23.50	23.74	23.97	24.21	24.44
13	24.68	24.92	25.16	25.40	25.64	25.88	26.12	26.36	26.60	26.84
14	27.08	27.32	27.56	27.81	28.05	28.29	28.54	28.78	29.03	29.27
15	29.52	29.77	30.01	30.26	30.50	30.75	31.00	31.25	31.50	31.75
16	32.00	32.25	32.50	32.75	33.00	33.25	33.50	33.76	34.01	34.27
17	34.52	34.77	35.03	35.28	35.54	35.79	36.05	36.31	36.56	36.82
18	37.08	37.34	37.60	37.85	38.11	38.37	38.63	38.89	39.15	39.41
19	39.67	39.93	40.19	40.45	40.71	40.97	41.24	41.50	41.76	42.03

H	0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
20	42.3	45.0	47.7	50.5	53.2	55.9	58.8	61.6	64.5	67.4
30	70.2	73.2	76.2	79.2	82.2	85.2	88.2	91.3	94.4	97.5
40	100.6	103.8	107.0	109.2	112.4	116.6	119.8	123.1	126.4	129.7
50	133.0	136.3	139.6	143.0	146.3	149.6	153.0	156.5	159.9	163.4
60	166.8	170.4	173.9	177.5	181.0	184.6	188.1	191.7	195.2	198.8
70	202.3	205.9	209.6	213.2	216.9	220.5	224.2	228.0	231.7	235.5
80	239.2	243.0	246.7	250.5	254.3	258.1	261.9	265.7	269.5	273.4
90	277.2	281.1	285.0	288.8	292.7	296.6	300.5	304.4	308.4	312.3
100	316.2	320.2	329.2	328.2	232.2	336.2	340.2	344.2	348.2	352.3
110	356.3	360.3	364.4	368.4	372.5	376.5	380.6	384.7	388.8	393.0
120	397.1	401.3	405.4	409.6	413.8	418.0	422.2	426.4	430.5	434.7
130	438.9	443.2	447.4	451.7	455.9	460.2	465.5	468.7	473.0	477.2
140	481.5	485.8	490.1	494.5	498.8	503.2	507.5	511.9	516.3	520.6
150	525.0	529.4	533.7	538.1	542.5	546.8	551.3	555.8	560.2	564.7
160	569.1	573.5	578.0	582.4	586.8	591.2	595.7	600.3	604.8	609.3
170	613.9	618.4	622.9	627.4	632.0	636.5	641.1	645.6	650.2	654.8
180	659.3	663.9	668.5	673.1	677.7	682.3	486.9	491.5	496.1	500.7
190	705.3	709.9	714.6	719.2	723.9	728.5	733.2	737.9	742.6	747.3
200	752.0									

power (column 4) are found at the points of intersection of the line *CD* of Fig. 5 with the proportionate-head values; and with the aid of Fig. 4 the intermediate values of proportionate head are found in terms of proportionate power or proportionate speed. The values of proportionate horsepower found are multipled by the value of the normal horsepower (2.04), giving the absolute *HP.'*, in column 5. Then each value of *HP.'* is multiplied by the $\frac{3}{2}$ power of its corresponding head (column 6).

Suppose this wheel is to be used in a small industrial plant that not only must have a constant speed, but also a constant power output under the varying head conditions. This, it must be understood, is a very rigid test of the wheel and calls for extreme flexibility. If we use as the constant power and speed required the value of the normal power and speed just found of 220 r.p.m. and 255 hp., then we simply read the efficiency along the line *AB* where this line crosses the proportionate-head values, again using Fig. 4 for intermediate points. These values are tabulated in column 7 and, considering the very severe requirements, show remarkable results.

If we are handicapped still further and are confronted with bad backwater conditions, we may often materially improve the situation for the lower heads by meeting a change in speed and power requirements and then "undergating" the wheel. In this matter we simply increase the overload capacity of the wheel, usually at a slight expense of maximum efficiency.

Take our present example: Let us use the speed and power corresponding to point *G* on the diagram as the constant values. *G* is at proportionate speed of 0.85 and proportionate power of 0.84. We then have a constant speed of 185 instead of 220 and a constant power of 215 instead of 225. And now reading along the line *EF* (drawn as was *AB* after shifting the proportionate-head scales so that 1 coincides with 0.84 and 0.85 old speed and power points) as before, along *AB*, the efficiency values are found as given in column 8 of Table 5. We still have a wheel capable of giving the 300-hp. maximum under normal head, but at the accepted reduced speed of 185 and an efficiency of about 76%.

Machines for Building Levees*

By J. R. Slattery†

SYNOPSIS—Great amount of Mississippi levee work made it imperative that machines be developed to take place of old man-and-mule methods. Description and costs of three types of machines.

The several floods of 1912 and 1913 in the Mississippi River resulted in a number of crevasses, which it was imperative to close prior to the time the next high water was to be expected. There was consequently placed on the market a large amount of emergency work—more than the contractors then in business on the river could handle. The result was that prices soared, until in 1913 the average price of levee work let by contract was 23.39c. in the Fourth District, 25.59c. in the First and Second Districts and 36.01c. in the Third District.[1] The average contract price of work in the Lower Yazoo District during previous years is shown by Table 1.

Recognizing the inability of existing team outfits on the river to handle economically the levee work made necessary by the great floods of 1912 and 1913, the Mississippi River Commission on Apr. 21, 1913, directed the district officers to give careful consideration to the possibility of using suitable mechanical appliances, either hired or purchased, with a view to securing a large output and a substantial gain in economy, and again on Nov. 21, 1915, directed the district officers "to make diligent inquiry into and study of the possibility of developing and acquiring suitable mechanical devices for use in the economical construction of efficient levees, with a view to their adoption."

TABLE 1. AMOUNT AND UNIT COSTS OF MISSISSIPPI RIVER LEVEES

Fiscal Year	Total Cu.Yd. Placed	Average Price per Cu.Yd., Cents	Fiscal Year	Total Cu.Yd. Placed	Average Price per Cu.Yd., Cents
1882–90	1,672,619	28.83	1902–3	841,589	17.21
1891–92	979,154	22.37	1903–4	1,905,842	17.24
1892–93	793,365	21.12	1904–5	1,040,708	17.40
1893–94	785,642	17.50	1905–6	429,632	19.75
1894–95	1,326,255	11.19	1906–7	305,507	21.89
1895–96	1,149,411	9.26	1907–8	918,504	20.55
1896–97	256,153	11.70	1908–9	986,674	25.31
1897–98	4,141,326	12.67	1909–10	303,878	22.08
1898–99	2,845,342	11.67	1910–11	906,761	20.34
1899–1900	1,840,340	17.30	1911–12	420,260	11.64
1900–1901	614,940	13.52	1912–13	1,072,305	25.80
1901–2	21,932	18.56			

*From "Professional Memoirs," Corps of Engineers, United States Army and Engineer Department at Large, May-June, 1916.

†Major, Corps of Engineers, United States Army, Vicksburg, Miss.

[1]The Mississippi River Commission is engaged in, among other things, the construction, extension and repair of levees from Cape Girardeau, Mo., to the Head of the Passes. The stretch of river is divided, for the execution of levee and other work, into four districts. The First District extends from Cape Girardeau to the foot of Island 40 (270 mi.); the Second from the foot of Island 40 to the mouth of White River (173 mi.); the Third thence to Warrenton, Miss. (214 mi.); the Fourth thence to the Head of the Passes, about 13 mi. from the Gulf of Mexico (453 mi.).

The standard specifications for levee construction provide as follows in regard to the borrow pits from which the material for the construction of levees is to be obtained:

BORROW PITS

For Levees Above Red River—Unless directions are given otherwise by the contracting officer, all earth must be procured from the river side of the levee. The location of borrow pits, whether on river side or land side of the levee, will be determined by the contracting officer for each section of the work advertised, and will be a part of the information furnished prospective bidders. When shown on maps or profiles submitted to bidders that earth cannot be obtained opposite stations, it must be hauled from places designated without extra compensation. In no case shall material be obtained within 40 ft. of the base of the levee on the river side or within 100 ft. on the land side. The side slope of the pit next to the embankment must not be steeper than 1 on 2 down to a depth of 3 ft.; from that point outward the pit may be deepened with a slope of 1 on 50 when on the river side of the levee and 1 on 100 when on the land side. On enlargement work no material will be taken from the old pits without special permission of the contracting officer, in which case the old pits that are not already down to the above specified limits below the original natural surface of the ground may be deepened accordingly; but in any case the material must be excavated and removed in such manner as will afford thorough drainage to the back of the pit and disfigure the land as little as possible. Any material taken below the specified depth must be replaced, and any borrow outside the limits of the pits as defined herein shall, unless direction has been given for such borrow by the contracting officer, be promptly replaced, or the same will be measured and the yardage deducted from the next estimate.

For Levées Below Red River—The same as above, except that in no case shall material be obtained within 20 ft. of the base of the levee on the river side or within 80 ft. on the land side. The side slope of the pit next to the embankment must not be steeper than 1 on 2 down to a depth of 3 ft.; from that point outward the pit may be deepened with a slope of 1 on 10 when on the river side of the levee and 1 on 100 when on the land side.

The generally smaller levees below Red River and the more liberal pit specifications for them made the selection of a suitable type of machine for their construction much less problematical than the selection of a machine for levees above Red River. The machine selected was the drag-line excavator, of the self-contained, revolving locomotive crane type, arranged to operate an orange-peel, clamshell or drag-line bucket at a radius of 90 or 125 ft.

Locomotive Crane Drag-Line Machines

The machines are mounted on four-wheel trucks and are provided with the necessary mechanism for performing the operations of traveling and hoisting, rotating and operating a bucket. Track is provided for the machines to work on during low-water stages and barges from which to operate them during high-water stages. The barges are likewise used to transport the machines from one job to another. The crews of the machines are housed on quarter-boats. The first machine with its barge, quarter-boat, track and equipment cost approximately $40,000. The second and third machines purchased are stronger and of greater capacity than the first one, and the last machine purchased has been so successful as to warrant its adoption with minor modifications as a type for work in the Fourth District—particularly below Red River. This type machine will handle a 3-yd. orangepeel bucket or a 5-yd. drag-line or clamshell bucket at a radius of 126 ft. from the center of the machine. The cost of machines of this type fully equipped and provided with a quarter-boat for the housing and care of crew and coal barge for supplying it with coal will be approximately $60,000. This figure does not include the cost of a barge from which to operate it during high stages and with which to move the machine from point to point. The cost of a steel barge suitable for this purpose is about $18,000. It will not be necessary to provide a barge for every machine. It is thought that with these machines it will be possible to place earth in the levee system, wherever rehandling is unnecessary, at field costs of from 5 to 10c. per cu.yd.—exclusive of the cost of clearing and grubbing when this is necessary. When material can be placed in the levee without rehandling, this type of machine is believed to be the most efficient for levee work. (See Fig. 3.)

Above the mouth of the Red River the levees become much higher than below the Red, and this fact, together with the more difficult pit specifications, necessitates much wider borrow pits and therefore greater reach on the part of the machines. Inasmuch as pits, when once dug, do not fill up again, this is true whether the work is enlargement of existing levees or the construction of new lines. The considerably greater reach necessary led to the selection of cableway machines for trial in preference to drag-line machines. Cableways were first operated in the Fourth District. The first one to be operated outside of that district was near Memphis. This cableway was of the slack-line type, was designed by C. G. Oglesby and put in operation by R. H. and G. A. McWilliams.[2]

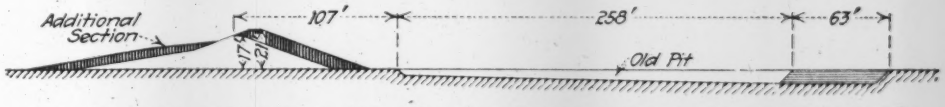

FIG. 1. AVERAGE CONDITION AND NECESSARY ENLARGEMENT OF LEVEES ABOVE GREENVILLE, MISS.

In the Third District the levees on the Mississippi side—except at the extreme lower end of the district—generally have banquettes, varying in width from 10 up to 20 ft. The banquettes have generally been added after the completion of what may be termed the main part of the levee. The main part of the levee has almost invariably been constructed from river-side pits, and the banquette generally, but by no means always, from land-side pits.

Above Greenville the average height of the levee system is 20.7 ft.; the average width of the existing river-side pits is 258 ft.; the average amount of material to be added to the existing levees to bring them up to the grade and section calculated to be necessary to contain safely such a flood as that of 1912 is 2,150 cu.yd. per station. In order to obtain this amount of material it is necessary to go out 63 ft. beyond the existing river-side pit. The outer edge of the resulting pit would then be 428 ft. from the center line of the crown of the existing levees. Even if this earth were all to be placed on the river side, drag-line excavators to do this work without rehandling would have to handle their buckets on booms approximately 200 ft. long. However, if the

[2]Described in "Engineering News," Sept. 10, 1914, p. 544.

enlargement is to be accomplished with the minimum of material, 610 yd. per station out of the total 2,150 yd. should be placed on the land side of the crown of the levee in order to widen and raise the banquette. This material can be placed cheaply by teams, if it is taken from land-side pits; but such pits are objectionable because they in a measure weaken the levee, take land capable of cultivation and mar the appearance of the land fronting on the levees.

When there is no banquette, the top of the levee can be readily cut down to the level of the new banquette and all the material then be placed on the river side, at the expense, however, of from 300 to 500 yd. per station extra; but when there is a banquette, it is seldom at the right height for the new banquette, and a satisfactory enlargement by this method can rarely be made.

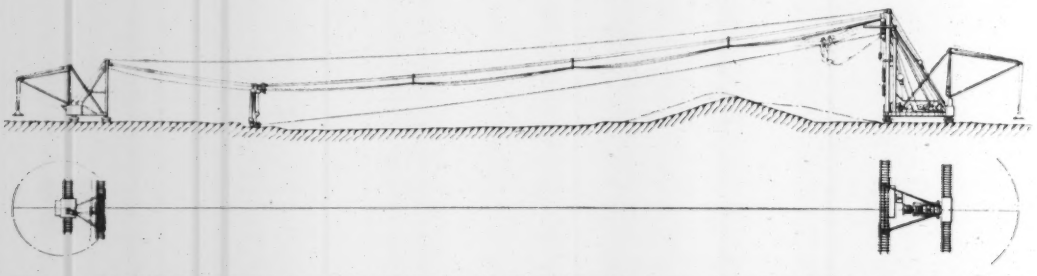

FIG. 2. LIDGERWOOD DRAG-LINE CABLEWAY EXCAVATOR USED IN VICKSBURG DISTRICT TO BUILD MISSISSIPPI LEVEES

A thin layer of earth must almost invariably be added to the banquette to bring it to the proper height, and thin layers of earth on the land side add little or nothing to the stability of a levee.

Taut-Line Cableway Type

A machine was therefore sought that would take all the material from the river side of the levee, beyond the limits of the old pits, and place it as needed either on the river side or land side. A taut-line cableway of the Lidgerwood type, equipped to handle a drag-line bucket, was thought to offer the greatest promise of meeting these conditions, and one was accordingly purchased by the United States and started to work at the extreme upper end of the Third District.

The first cost of a machine of this type, erected and equipped, is about $45,000. Its general features are shown in Fig. 2. It has a clear span of 662 ft. The towers are of steel, 85 and 45 ft. high respectively, and support between them a 2¼-in. cable. On this cable travels a carriage which carries a 3-yd. drag-line bucket. The carriage is moved back and forth by means of an endless line, operated by one of the drums of the main engine. The bucket is loaded by means of a cable which leads from the front end of the bucket to another drum on the main engine, and is lifted by means of a cable which is attached to the bail of the bucket and runs thence around a sheave on the carriage, over a sheave at the top of the head tower and thence to a third drum on the main engine.

A pull of 50,000 lb. for loading the bucket is developed by means of a special engine, which operates a haul-down rope which runs from the drum of this engine around one sheave of a double tandem block (the second sheave of which rides on the drag or loading line) and thence to the frame of the tower.

The bucket is dumped at will by means of a haul-down rope which pulls down on the dumping [illegible] attached to the back of the bucket, and running thence to the conveying drum, thus accelerating the movement of the rear of the bucket relatively to that of the front end and consequently causing the back end of the bucket to lift and the material to dump. This haul-down rope is operated by means of a piston and steam cylinder. A special engine is provided on the head tower to move the same. The tail tower is moved by means of a friction drum operated by the conveying line.

Derricks were originally provided on both head and tail towers to pick up track from behind the machine. This method of handling proved satisfactory on the tail tower, three men being able to handle the track without difficulty. The long boom of the derrick on the head tower made this derrick unwieldy to handle by hand, and its use was therefore abandoned and the track thereafter dragged forward by means of the moving engine. An improved system of track handling is contemplated to be used on both towers during the coming season.

The frictions and brakes are all operated by compressed air.

The crew of this machine consists of one foreman rigger; one operator; one rigger's helper; one engineman; one fireman; one signalman; eight laborers (trackmen), three on tail tower and five on head tower; three laborers (dressing levee); three teamsters (ploughing, dressing levee and hauling supplies).

Table 2 gives the results of the season's (1915) work with this machine.

TABLE 2. WORK DURING 1915 BY LIDGERWOOD LEVEE BUILDER

Month	Yardage Placed During Month	Cost per Cu.Yd.	Yardage Placed Since Work Began	Average Cost per Cu.Yd
April	5,211	$0.1850	5,211	$0.1850
May	13,239	.1386	18,450	.1518
June	20,050	.1548	38,500	.1534
July	23,850	.1126	62,350	.1378
August	19,050	.1490	81,400	.1404
September	2,600	.7047	84,000	.1579
October	22,800	.1041	106,800	.1464
November	28,500	.1108	135,300	.1389
December	18,600	.2253	153,900	.1494

The very high costs of September were due to practically the entire month having been lost on account of high water, wet pits and delays incident to renewing the main cable—the latter due purely to bad management. The high cost of December was due to a flood and to the necessity of stopping work some days before actually drowned out, in order to prepare for a prolonged high water. Shortage of coal and the holiday season also cut down the output of this month. The past season was an exceptionally bad one for levee work, and progress was retarded throughout by wet pits.

Various parts of the machine which proved unequal to the strain it was desired to put upon them have been or are being replaced or strengthened by the Lidgerwood Co., and it is expected that delays during the coming season will be fewer and that the average cost per yard will be much less. It is thought that with proper management this machine should be able to build or enlarge levees at field costs ranging from 8 to 14c. per cu.yd. As the material falls into place from heights varying from 10 to 40 ft. it is believed that this type of machine builds a particularly good levee.

Slack-Line Cableway Type

The manufacture of slack-line cableways of the type developed by Mr. Oglesby has been taken up by the Bucyrus Manufacturing Co., which, after a careful study of Mr. Oglesby's first machine, built a greatly improved machine of this type for H. B. Blanks & Co. on their contract for levee enlargement below Greenville. It is understood that the cost of this machine, erected and equipped, is about $40,000. It has a clear span of approximately 450 ft. The head tower travels on the banquette, which has to be first enlarged by teams. The machine therefore builds only the river-side enlargement, which on this work averages about 1,800 yd. per station. The bucket is of 8-yd. capacity. The machine was not completed until about the middle of October, 1915. Between this time and the end of the year its output was as follows: October, 10,340 cu.yd.; November, 31,077 cu.yd.; December, 28,083 cu.yd.

On account of its working on higher ground and considerably lower down the river than the Lidgerwood machine, it was not compelled to stop work on account of high water until Jan. 8, 1916.

The crew of this machine comprises two foremen and riggers, two head-tower operators, one tail-tower operator, one oiler, one fireman, ten laborers (trackmen and dressers), one team and teamster.

The unit cost of work accomplished by the Bucyrus machine has been considerably less than that accomplished by the Lidgerwood machine, but it is not yet safe to draw final conclusions as to the relative merits or economy of these two types of machines. The conditions under which they have been working have been very different. The Bucyrus machine worked only during the best part of the last working season, whereas the Lidgerwood worked the entire season, was hampered most of the time by wet pits and was completely drowned out for the best part of one month. The Bucyrus has moved dirt an average distance of 310 ft., whereas the Lidgerwood has moved it an average distance of 412 ft. Building only the river-side enlargement, the Bucyrus machine has had only about 6,000 sq.ft. of levee to dress per station, whereas the Lidgerwood, which builds the banquette also, has had 17,200 sq.ft. per station to dress. The Bucyrus has had to haul its coal and supplies from ¼ to 1 mi., whereas the Lidgerwood has had to haul its coal and supplies from 4 to 6 mi. The Bucyrus has been working in a locality where labor is much more plentiful than in the locality where the Lidgerwood has been working. The Bucyrus has not been working long enough to necessitate much expense in the way of repairs or renewals.

It seems, however, safe to predict that the Bucyrus machine will build river-side enlargement or new levees at field costs ranging from 8 to 14c., exclusive of the cost of any grubbing and clearing that may be necessary. Its designers claim that it can be designed to enlarge existing banquettes or build banquettes behind existing levees and have constructed two machines so designed for Mr. Oglesby's use in the Second District. As these machines have not yet started to work, it is not safe to predict how successful they will be on banquette work.

Considerable difficulty has been experienced in getting the contractor operating the Bucyrus machine to leave pits conforming to specifications. This is thought to be principally due to the fault of the contractor, but is also in some measure due to the difficulty of controlling the bucket while being loaded and dragged through the pit to the levee. While the Bucyrus machine compacts the material to a certain degree as a result of dragging the heavily loaded Bucyrus bucket over the levee and because of the bucket's slipping back slightly just before dumping and thus ironing out, so to speak, the new material placed in the previous load, the Lidgerwood machine is considered to compact the material much more tightly as a result of the height from which the material falls.

FIG. 3. BUCYRUS DRAG-LINE MACHINE WORKING NEAR BATON ROUGE

The locomotive crane drag-line excavators have already reached a high stage of development, and improvements in their design will be slow from now on, but considerable further improvement is anticipated in cableways. The Bucyrus cableway is a long step in advance of the machine constructed by Mr. Oglesby. Experience with the Lidgerwood cableway has suggested many desirable improvements, and it is thought that the next Lidgerwood machine will be quite as much of an improvement on the one now in use as the Bucyrus purchased by Blanks & Co. is on the machine first built by Mr. Oglesby.

Most of the levee work on the Mississippi River will undoubtedly be done by levee-building machines from now on, at costs materially lower than it would be possible to do it by teams.

Land-Clearing and Stump-Pulling methods are to be demonstrated throughout the northern section of Wisconsin by means of an exhibition train operated by the University of Wisconsin over the lines of the Chicago & Northwestern Ry. and the Chicago, St. Paul, Minneapolis & Omaha R.R. The train will be equipped with different makes of hand- and power-operated stump pullers and with apparatus for blowing up stumps. Stops will be made at a number of towns, according to a schedule advertised in advance, and stump-removing operations will be exhibited. The work will be under the direction of Carl Livingstone, of the Agricultural Engineering Department, University of Wisconsin.

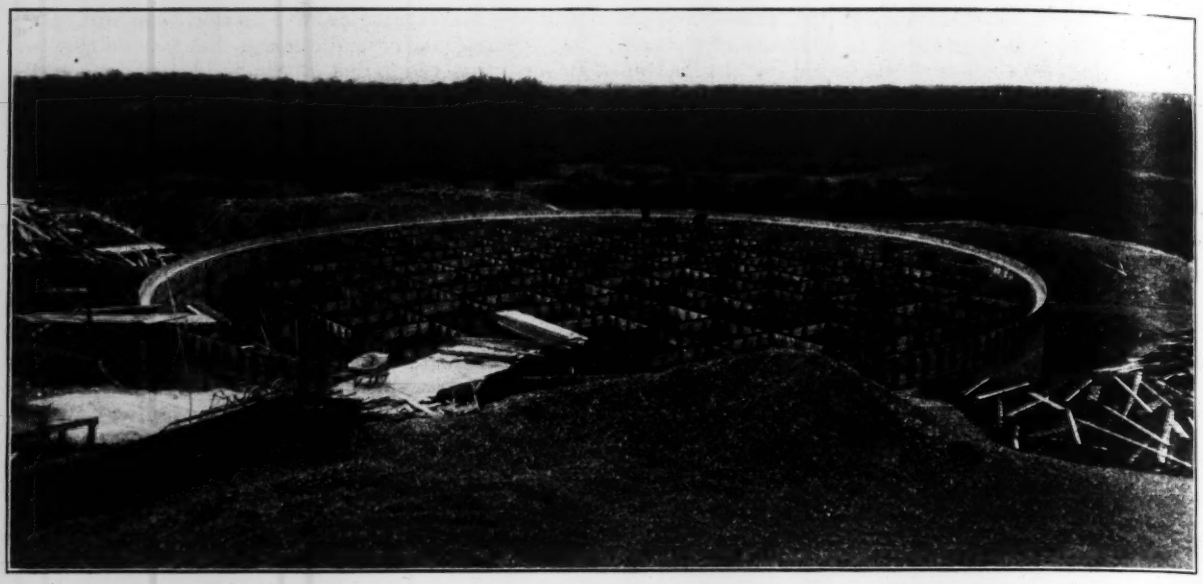

FIG. 1. COVERED REINFORCED-CONCRETE EQUALIZING AND STORAGE RESERVOIR FOR HIBBING WATER-WORKS

Covered Reinforced-Concrete Reservoir, Hibbing, Minn.

By R. E. McDonnell*

The water-supply for Hibbing, Minn., a village of about 12,000 population, is procured from a series of drilled wells. Most of the wells are so located that the water flows from the delivery pipe at the wells to a large equalizing and storage reservoir. A temperature of 50° F. is reached at Hibbing in winter, and as the wells are pumped intermittently, it is necessary to protect the water in the reservoir from freezing.

The reservoir is a reinforced-concrete tank with an inside diameter of 115 ft., is 14 ft. deep and holds about 1,000,000 gal. It is built on a glacial deposit of sand and gravelly soil, and to make it absolutely water-tight was necessary in this sandy formation.

The floor of the reservoir consists of two layers of 4-in. concrete slabs, between which is a layer of asphaltic waterproofing material. The roof consists of a 5½-in. reinforced-concrete slab, supported on 10x12½-in. reinforced-concrete columns, with footings enlarged to give an area sufficient to carry the reinforced-concrete roof slab, 18 in. of earth cover and snow which sometimes reaches a depth of 4 to 6 ft.

The roof of the reservoir will be sodded and arranged with flower beds in keeping with the other surroundings of the power house.

It is anticipated that a small amount of sand will be pumped from the wells. Accordingly, a sand trap has been provided to keep the sand from reaching the 24-in. suction line leading to the high-service pump. A glass-covered inspection chamber provided with electric lights affords opportunity to observe the water in both the sand trap and in the reservoir. The electric lights are embedded in concrete with special conduits for lighting under water.

The double floor slab is laid in blocks 8½ ft. square, with joints staggered and filled with asphaltic preparation. The concrete was waterproofed, and tests for leakage were made after the completion of the reservoir. It was impossible to detect any leakage after the reservoir had been filled for 48 hr.

In the construction of the reservoir the vertical walls were poured at one pouring from a circular track laid just above the vertical walls, and from this track the

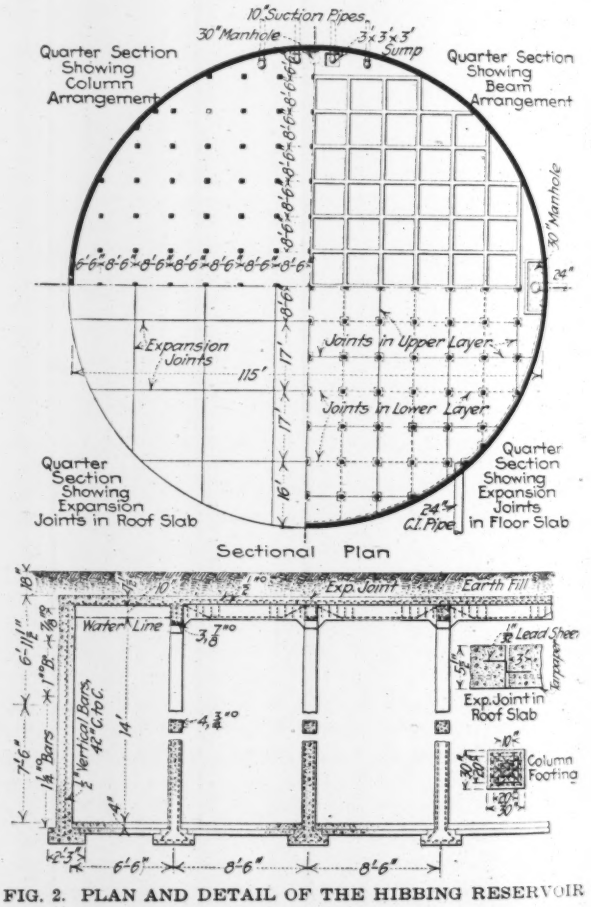

FIG. 2. PLAN AND DETAIL OF THE HIBBING RESERVOIR

*Of Burns & McDonnell, Consulting Engineers, Kansas City, Mo.

concrete was dumped from wheelbarrows into the vertical forms. The roof was likewise poured continuously.

The reservoir was a part of the general construction contract of Layne & Bowler Co., of Houston, Tex., but was built by subcontractor E. W. Coons Co., of Hibbing, Minn. It was designed and the construction was supervised by Burns & McDonnell, consulting engineers, Kansas City, Mo.

Legal Decisions of Interest to Engineers

Written Contract for Construction Work Supersedes Verbal Contract—Where the parties to a contract for railroad construction work did not execute the written contract until a few days before the work was completed, the written contract nevertheless governs and supplants any verbal declarations concerning the nature of the work, and that plaintiff would make a profit, which declarations were not inserted in the written contract.

The Arkansas Supreme Court came to the foregoing conclusion in the case of Robertson vs. Sharp Contracting Co. The Robertson Co. had the contract for constructing 50 mi. of railroad for the Malvern & Camden Railroad Co. The understanding was that certain of the work was to be classified as "wet excavation." Shortly before the completion of the railroad the parties reduced the terms of the contract to writing. The written contract, however, contained no "wet excavation" classification.

The question before the court was whether the construction company was bound by the written contract omitting the "wet excavation" classification or could recover for wet excavation work, as was the verbal understanding. The court held that the written contract must govern and supplant any verbal declarations concerning the work. (Robertson vs. Sharp Contracting Co., 183 S. W. 754.)

Buyer of Machinery May Refuse to Pay Note Where Machinery Does Not Fulfill Guarantee—Where a manufacturer of dredge machinery in making a sale accepts the purchaser's promissory note as part payment for the machinery, the purchaser may refuse payment of the note if it is later found that the machinery does not meet the warranty as to its fitness to perform the work for which it was sold. (Murray Co. vs. Palmer, 154 Pac. 1137.)

Contractor's Delay Through "World-Shared Ignorance" Is Not an "Unavoidable Cause"—By contract with the United States Government, the Carnegie Steel Co. engaged to furnish to the Ordnance Department armor plates, some of them to be 18 in. in thickness. Certain plates were to be delivered on specified dates for the ballistic test. The contract provided for liquidated damages for delay in delivery other than through "unavoidable causes, such as fires, storms, labor strikes, action of the United States," etc. The company failed to make timely delivery, and the United States retained part of the purchase price as liquidated damages. A demurrer to a petition in the Court of Claims for the recovery of this balance was sustained except as to a small part of the money retained.

It is claimed by the company that the delay was by "unavoidable cause." No hard-faced 18-in. armor plate had previously been manufactured in this or any other country, and no information was obtainable respecting the processes involved. The company made a single plate to determine the proper treatment and processes, but delayed the manufacture of the others, from which plates for government tests were selected, until it was demonstrated that the first plate answered all requirements of the contract.

Justice McKenna, in delivering the opinion for the United States Supreme Court, said: "It would seem that the very essence of the promise of a contract to deliver articles is ability to procure or make them. But claimant says its ignorance was not peculiar, that it was shared by the world, and no one knew that the process adequate to produce 14-in. armor plate would not produce 18-in. armor plate. Yet claimant shows that its own experiments demonstrated the inadequacy of the accepted formula. A successful process was therefore foreseeable and discoverable. And it would seem to have been an obvious prudence to have preceded manufacture, if not engagement, by experiment rather than risk failure and delay and their consequent penalties by extending an old formula to a new condition." (Carnegie Steel Co. vs. United States, 36 Sup. 342.)

Town Planning in Canada

By Thomas Adams*

What is known as "city planning" in the United States and "town planning" in Canada has not received the attention it should have received from the engineering profession. The problems that have to be solved in planning schemes are not only largely of an engineering character, but it is the engineer who has to prepare the foundation on which all schemes have to be built up. The under-emphasis which has been placed on the engineering aspects of the problem has been in part due to the indifference of engineers themselves. They will have to overcome this inertia in justice both to the movement and to themselves.

In the States men like Nelson P. Lewis, of New York, have taken a prominent part in city planning, but a wider interest is needed. In Great Britain practically the whole of 152 schemes in course of preparation are in the hands of municipal engineers. This does not mean that the architectural viewpoint is overlooked, but merely that the preliminary stages of planning require engineering and local knowledge which the architect does not possess as a general rule. The designing, placing and grouping of buildings, and even the layout of a park system have to follow the design of the street, drainage, and transportation systems, and owing to the necessarily individualistic method that has to be followed in the erection of buildings on separate lots, it is usually found that it is not practicable to apply any conscious design to large groups of buildings. On the other hand, the street system and transportation is customarily a matter for public control and comprehensive treatment.

Town planning in Canada is being placed in the hands of engineers and architects acting in collaboration. The Commission of Conservation is the advisory body for the whole Dominion and has its engineering and public-health experts. Housing and town planning are dealt with as twin problems, and the primary objects of all

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schemes are to secure (1) proper sanitary conditions, (2) convenience for traffic and (3) amenity or agreeableness in the development of land. Hitherto the owners of real estate have been allowed to plan their land with the sole object of private gain and greatly to the inconvenience of the public. More coöperation is needed in the interests of such owners themselves, and eminently in the interests of the public welfare.

The passing of legislation to give city, town and municipal councils powers to prepare proper schemes is the first essential. Four provinces out of nine in Canada have now town-planning acts, and the other five provinces are likely to have acts on their statute books within another year. Nova Scotia has a compulsory act which requires that the whole province must be the subject of a scheme of planning within the next three years. These schemes have to provide for:

1. Main thoroughfares (up to 100 ft. wide).
2. Building lines (40 to 60 ft. from the center of each street of whatever width).
3. Narrow residential roads of short length (24 to 66 ft. according to traffic requirements.)
4. Reservation of residential and manufacturing areas.
5. Restriction on extent of lot to be built on and number of dwellings to each acre.
6. Preservation of features of natural beauty and restriction on nuisances, billboards, etc.
7. Provision of air space surrounding and within dwellings, etc.

The street system of each municipality has to work in with the general highway plan of the whole province. Wide main thoroughfares have to be arranged by "give and take" arrangements with real-estate owners by which complementary narrow streets are permitted in places where traffic requirements are light. Thus these main thoroughfares have not to be made at the cost of the general taxpayer, but by a sensible variation in width under a scheme. Fixing widths of streets by ordinance or by law is always unsatisfactory because such ordinances have to be specific in character and general in their application. Town-planning schemes allow discretion to be used and every street or other feature to be considered on its merits.

Of the schemes under consideration in Canada the largest, and the first statutory scheme to be started, is that of St. John, N. B., which deals with about 20,000 acres, of which half are inside and half are outside the city. Although the application to prepare this scheme carried with it the power of the City Council to restrict real-estate development, and to prevent any owner from contravening the proposed scheme, before it was prepared, only one owner raised an objection and was overruled.

A plan has been prepared by a special commission for the reconstruction and replanning of the capital city of Ottawa. The expert advisers were Mr. Bennett, architect, of Chicago, and Mr. Cousins, civil engineer, of Toronto.

Another development in Canada for which the Commission of Conservation is responsible is the formation of a Civic Improvement League for the Dominion. The object of this League is to educate public opinion so as to use the new town planning powers. A strong engineering committee of the League has been formed under the chairmanship of the Ontario Deputy Minister of Highways to prepare a report on Highways and Transportation. The good-roads movement has also linked itself up with the town-planning movement, so that the planning of highways is being considered side by side with methods of construction.

Extensometer Tests on Three Types of Concrete Floors

The City of Seattle, under its building code which went into effect in August, 1913, can require full-size tests of completed reinforced-concrete floors. Under this provision of the building law the city has within the past year tested three types of reinforced-concrete floors, namely, the Turner "Mushroom" floor, the Cauldron "Akme" floor, and an ordinary girder, beam and slab floor. The tests, which were made with extensometer readings of steel and concrete deformations, form the main part of the January-February, 1916, issue of the "Proceedings" of the Pacific Northwest Society of Engineers.

The papers are entitled "Tests of the Bell St. Warehouse of the Seattle Port Commission" by Almon H. Fuller and Charles C. Moore, of the University of Washington, and "Tests of the Sears-Roebuck Building and of the New Courthouse of King County" by D. E. Hooker, Assistant Superintendent, Seattle Building Department. In both the papers the data are given very fully, so that any one who is interested in the subject should obtain a copy of the Proceedings from the secretary of the society at Seattle. A brief description of the main facts brought out in the papers will be given below.

Tests of Turner Flat-Slab Floor

The Turner flat-slab floor tested was on the third floor of the Bell St. warehouse built by the Seattle Port Commission. This floor, designed according to the well known disposition of steel used by C. A. P. Turner, consisted of two wall panels 20 ft. 9 in. by 19 ft. 4 in. and two adjacent interior panels 20 ft. 9 in. by 20 ft. The design called for a 10¾-in. slab with four-way bands 9 ft. 6 in. wide, each carrying seventeen ½-in. rounds placed 10 in. from the top of slab in the middle of the span and 8 in. from the bottom at the column capital edge. The outer ring of the spider is a ¾-in. round, 9 ft. 6 in. in diameter and the inner ring a ⅞-in. round 6 ft. 9 in. in diameter at the interior columns and 5 ft. 6 in. at the wall column. The column capital is about 6 ft. in diameter at the floor. The floor was designed for a live-load of 500 lb. per sq.ft. and a dead-load of 144 lb. per sq.ft. Loadings of sheet tin in boxes were placed in increments of about 265 lb. per sq.ft. on all four panels until a final total test load of 700 lb. per sq.ft. was placed all over the four. In taking off the load a part was left on the wall panels while the other two were unloaded but at no time were the wall panels fully loaded while the interior panels were empty.

At the time of the test the floor was 113 days old (December-April) and there appears to be a controversy as to the condition of the concrete. The building department claimed that while the weather was cold when the slab was poured and remained damp and chilly for a considerable part of the time thereafter, still the conditions were not so adverse but what the concrete should have at least attained the strength it would under best conditions in 28 days, which is the time called for a test under the provisions of the building law. Mr. Turner's representative, however, claimed that the concrete was subjected to such low temperatures that it could not be expected to attain a proper strength in the period before the test. On account of this controversy and also on

account of some others which will be noted hereafter, the building department has announced that it will make another deflection test on some other portion of the building if possible.

Gage points were put in the steel and concrete in the upper and lower parts of the slab before the floors were poured and readings were made upon these points. At the same time, deformation tests were taken on concrete cylinders poured at the time the concrete of the floors was poured and also on concrete cut from the floors. The moduli of elasticity obtained from these small pieces were used in determining the stress in the concrete of the building. Tests on steel were also used to determine the moduli for the steel in the building. It should be said that the determination of the moduli for the concrete is none too conclusive.

The facts regarding the stresses determined are briefly stated in the paper as follows:

In the mushroom head of the interior columns the direct stresses in the ring and radial rods are within reasonable limits. In the wall columns the stresses are high in the concrete on the inside, but reasonable in amount in the vertical steel. A tension, however, approximating ordinary working stresses occurs in the outside rods. The stresses in both steel and concrete are within reasonable limits in the interior panels, but abnormally high in the wall panels, particularly the concrete stresses immediately adjacent to wall columns. The deflection of the wall panels exceeds that in the interior panels by more than 50% under a uniform load. A load only on wall panels induces greater stresses than a load of the same intensity on the adjoining panels as well.

In figures, the deformations show that the steel stress near the wall columns in some rods is as high as 50,700 lb. per sq.in. where the yield point of the steel is about 50,000 lb. and that concrete stress near the column is at least as high as the breaking stresses in the concrete tested; that is, 2,000 lb. per sq.in. All of this is under an external load of 700 lb. per sq.ft. when the designed load was 500 lb. per sq.ft. In addition, a number of hair cracks became visible under the second increment of load. The deflection under maximum load was 0.81 in. in the wall panel, recovering to 0.38 in. in three days and 0.41 in. in the interior panel, recovering to 0.14 in. in three days.

In discussion of the paper the designer, A. O. Powell, stated that while he was responsible for the selection of the system and for the original design, the work was entirely under the direction of the port of Seattle's engineers and that he had no control whatever over construction. The high stresses found were due mostly, in his opinion, to the testing of the slab before it had reached its final set. He also called attention to the fact that the steel in many cases was far out of place. In places where it was designed to be 8 in. above the bottom of the slab, it was found to be 5 in. On this account he claimed that the designed load should not be used in a comparison with the test loads.

Tests of Akme Flat-Slab Floor

The Sears-Roebuck Building is nine stories high, all the floors being built on the well-known "Akme" two-way system sold by the Condron Co. of Chicago. Tests were made over four panels, two interior and two wall panels, 52 days old (April-June). The panels were 20x20 ft. on the interior and 20x18 ft. at the wall, and the slab is 8 in. deep at the middle and 14 in. deep at the depressed head. The floor was designed for 200 lb. per sq.ft. live-load, and a dead-load of 125 lb. per sq.ft., although at the time of test the dead-load was only about 100 lb. per sq.ft. as the cinder fill was not in place. Test loads consisted of increments of about 200 lb. bringing the maximum up to 550 lb. per sq.ft., which it will be noted is about twice the design plus the dead load. These tests were made by the extensometer method, although the test points had not been placed during the construction of the building. Moduli of elasticity determined in the Bell St. warehouse test were used for the concrete and the steel. Under this arrangement the floor showed up very well in practically all points, the greatest steel stress being 27,300 lb. per sq.in. at the top of the slab; the greatest stress in concrete, about 1,000 lb. per sq.in. Maximum deflection was 0.48 in., with recovery approximating 100%.

The one important fact stated by Mr. Hooker in regard to the Sears-Roebuck Building was the effect of the eccentricity of load on the wall columns. According to Mr. Hooker, this effect is of such marked extent that it should always be taken into consideration in flat-slab design. Either flat-slabs at walls should be carried by spandrel beams, the wall panels being considered freely supported at the walls, or the columns should be sufficiently reinforced to care for the eccentric moment otherwise developed. It may be noted that the tested floor was next to the top floor of the building and the column was therefore carrying in addition to the test load only one floor slab and a roof slab, so that it did not have the restraining effect of an upper load which would be characteristic of a lower column. Also the brick walls and the spandrel beams were not in.

Tests of Plain Beam and Slab Floor

The new Court House of King County was an ordinary beam and girder reinforced-concrete building. The test area was one panel wide and three panels long, extending across one wing of the building on the fourth floor, and when tested was 128 days old (February-June). Remarkably small deflections and stresses were found. The maximum deflection noted under the final test load was little more than ¼-in. in the center of a 31-ft. span. The greatest observed stress in the center of the long span was somewhere in the neighborhood of 6,000 lb. per sq.in. in the steel, and 450 lb. in the concrete. These were stresses occasioned by maximum live-load which amounted to 1.2 times the designed live and dead loads. The design load was 65 lb. per sq.ft.

Mr. Hooker says concerning this test:

One fact stands out distinctly; that is, that slab action or shear or arching, or whatever one might call it, distributes stress far beyond the loaded width of floor out into the adjacent panels, and that concrete tension and arch action in the direction of spans reduce stresses very materially. If this were not true, the observed stresses would seem to refute the accuracy of assumptions which are used constantly in design. I believe these and similar field tests emphasize a fundamental difference between so-called flat-slab designs and ordinary designs consisting of slabs, beams and girders. The latter are based on mathematically demonstrable theories of beam action. When executed in a building we often find upon test actually much lower stresses than our theories indicate. Why? Because we gain an additional factor of safety from arching, concrete tension, and the tendency of adjacent panels to assist the loaded panels. On the other hand, flat-slab designs are worked upon empirical formulas, not mathematically demonstrable but worked out from observation of the action of test panels in completed buildings, and advantage has already been taken of arching, concrete tension, etc. Hence while we find lower stresses than we expected in the girder and slab floor, we often find as high or higher stresses than we expected in the four-way flat-slab floor.

Field and Office

Replacing Timber Bridge with Steel-Girder Spans

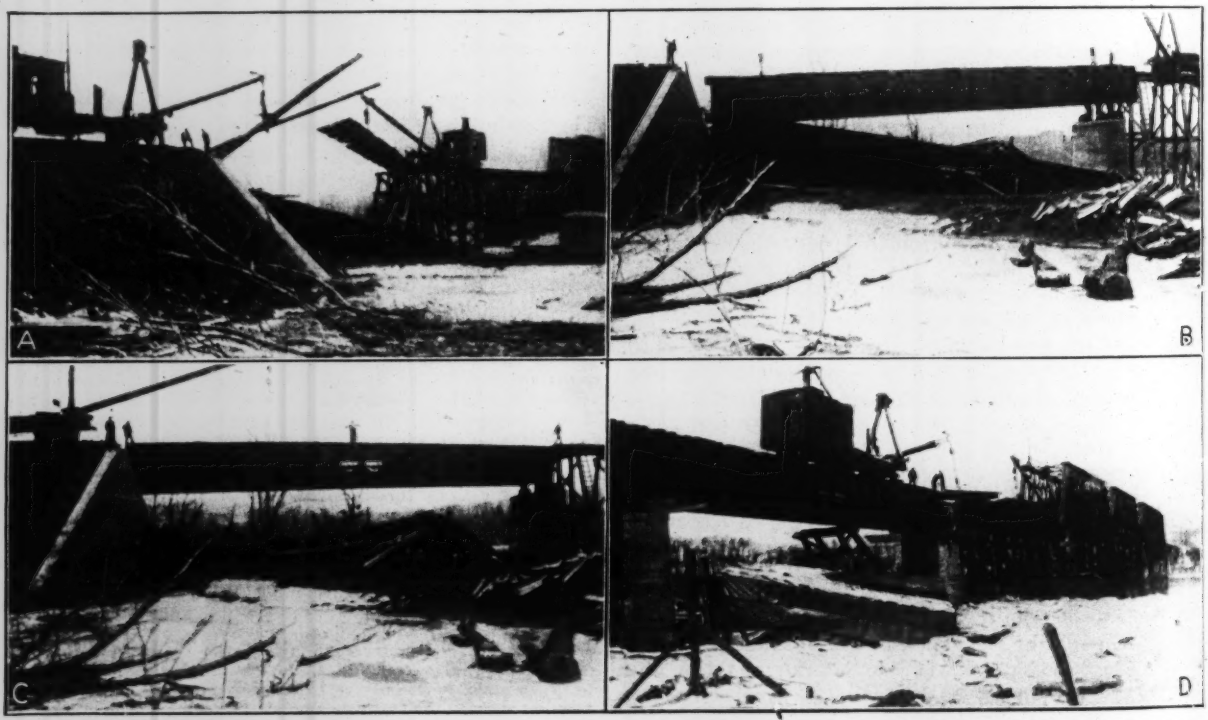
The Fort Dodge, Des Moines & Southern R.R. (a 120-mi. electric railway) crosses the Des Moines River at a point about 6 mi. west of Boone, Iowa. The original structure, built in 1903, before the road was electrified, was a wooden bridge consisting of one 150-ft. and two 104-ft. Howe truss spans, with 692 ft. of trestle approaches. The new bridge will consist of four 96-ft. and three 103-ft. steel deck-girder spans, the remaining 357 ft. being filled. Three of the 96-ft. spans were placed in the fall and winter of 1914-15, and three of the timber spans are still in place. The progress of the work is shown in the accompanying views.

The concrete work on the abutments and piers was done by company forces. The equipment consisted of a ⅝-yd. mixer and a timber tower with a steel boom to support the chute. The boilers supplying the mixer and hoist engines were interconnected. Water for boilers and mixer was drawn from the river by means of a steam jet fitted with a Y connection, which could deliver water either to boilers or to mixer.

The gravel was hauled to the site in ballast cars and dumped over the side of the bridge at a point convenient to the work. The cement was chuted from the cars on the bridge above to a platform at the cement shed.

The original plans called for 700 cu.yd. of concrete in the abutment and 200 cu.yd. in each of the three piers, but the footing of the abutment was placed about 4 ft. lower than at first intended, thereby increasing the yardage to 1,450. The force consisted of two enginemen, two firemen, one man operating the hopper, one man wheeling cement to the mixer, three men wheeling gravel, and one man spading concrete as it was dumped into the forms. Four men were used on the forms, and from eight to ten men were excavating for the piers while the concrete of the abutment was being placed. With this force and equipment 80 cu.yd. per day was placed when no delays were incurred. The average for the total number of working days was 60 cu.yd. per day.

The filling of 280 ft. of the pile approach on the south end was started Nov. 25. The average height of this fill was 25 ft. The material was hauled from the company's gravel pit, about 2 mi. from the bridge. The equipment consisted of a Vulcan 2-yd. steam shovel, 40-ft. side-dump gondola ballast cars pulled by a 35-ton electric locomotive, and a standard gravel plow with a 1½-in. cable. The plow cable was coupled to the engine by a Goodman wrecking hook, which after trials of other devices was found the most satisfactory for this purpose. This work was finished Dec. 8; 8,000 cu.yd. was placed by this method in the ten elapsed working days.



REPLACING A TIMBER-TRUSS BRIDGE WITH PLATE-GIRDER SPANS, FORT DODGE, DES MOINES & SOUTHERN R.R.

A—Girders for new end span landed on the ground and two derrick cars removing deck of old trestle. B—Placing the first girder with two derrick cars; note sway bracing on girder. C—First girder span completed. D—Two derrick cars setting girders of third span in place, girders temporarily outside of normal position, resting on cap timbers on piers. Three of the old timber spans still standing.

The work of steel erection was done by contract in February, 1915. To handle the girders the contractor used two derrick cars, one of 75-ton and one of 50-ton capacity. The plan first proposed was to build falsework bents upon which to land the girders; but after only two bents had been driven, the ice went out, carrying the bents with it. This plan was then abandoned, and the girders for two spans were first landed on three 8x16-in. stringers, 32 ft. long, placed flatwise across the piers. For the end span, the girders were placed on the ground.

The old wooden deck was then cut directly over the piers and halfway between them. A derrick at each end of the span picked up half of the deck and carried it out to the fill at the end of the bridge. Then with a derrick at each end the girders were landed in place. These changes were made, the new track laid and traffic resumed in three hours. One span was erected each day, all the steel being placed in three consecutive days.

The work was done under the direction of the Maintenance-of-Way and Bridge and Building Department: L. E. Gagnon, Superintendent of Maintenance-of-Way; R. L. Cooper, Chief Engineer. The late W. M. Hughes, of Chicago, was consulting bridge engineer. The Farrell Construction Co., of Chicago, had the contract for the steel erection.

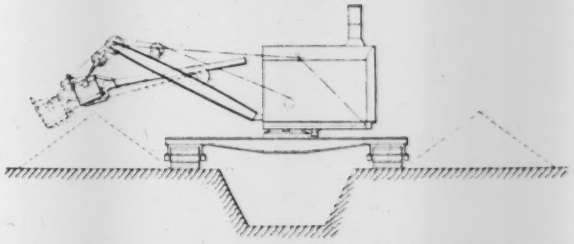
Power Shovels for Trenching

Practically any power shovel can be rigged to dig a trench. The best one for such service is the one that requires the shortest time for set-up, can handle the most material in a given time and costs least for repairs. Ordinarily a steam-shovel boom is maintained at a definite angle by means of hogrods. In the past six months at least two manufacturers have concluded that an arrangement for raising and lowering the boom has advantages over the rigid boom rods. This involves the use of an extra drum installed on the stayframe and geared to the main hoisting drum. A ⅝- to ⅞-yd. shovel—the popular size for small contractors—is generally equipped with a 17-ft. boom and dipper sticks 18 to 19 ft. in length. In order to go down 14 ft. or deeper, this boom and dipper handle have to be replaced by a boom 20 ft. long and a dipper handle 22 ft. in length.

With a boom raising and lowering device the extra reach required for deep digging is obtained by dropping the boom a few feet. The Osgood Co., of Marion, Ohio, which employs this arrangement except where the contractor specifies hogrods, cites a recent case where one of its Model 18 shovels, equipped with this device, excavated a sewer trench 46 in. wide, 15 ft. deep and 150 ft. long in a 9-hr. day. The material consisted of very dry and hard clay mixed with boulders. In another instance a similar machine excavated a sewer trench 17 ft. deep, 9 ft. wide and 76 ft. long in 7½ hr., loading one half of the material in wagons and depositing the other half on one side of the trench.

Like other shovels, this model may be equipped with a ¾-yd., a ⅝-yd. or a ½-yd. dipper, depending upon the nature of the material encountered and the kind of work to be done. For sewer-trench work the manufacturer recommends a special narrow dipper with the hitch on the back. In extremely deep work this type of dipper greatly reduces interference in filling. The dipper is generally fitted with five teeth, three in front and one on each side, the side teeth serving to draft the dipper into the side banks of the trench. A cutting lip is available (instead of teeth) wherever the character of the soil suggests its use.

The raising and lowering arrangement is, of course, not an exclusive feature of any shovel. All makers have



SMALL REVOLVING STEAM SHOVEL MOUNTED ON WIDE TRUCK FRAME EQUIPPED WITH CATERPILLARS

This arrangement—Osgood 18 fitted with boom raising or lowering device—is applicable naturally to soft ground, and avoids planking or artificial foundations. It is effective in excavating lateral ditches and in deepening or widening old channels.

used it in special instances, such as where the shovel is equipped with an exceptionally long boom for grab-bucket work, for locomotive-crane or derrick service, or for work in cities where the shovel must pass under overhead obstructions. If the contractor is willing to pay the extra cost occasioned by this device (about $150 in the case of small shovels), any shovel manufacturer can accommodate him. That the expense is worth incurring where there are trenches to be dug with steam shovels cannot be denied.

Red-Lead Finishing Coats

Since the Havre de Grace bridge test showed three-coat red-lead work in first and second place for endurance there has been some attention given by bridge engineers to the use of red lead for finishing coats. The most interesting recent case of red-lead top coats is the heavy plate-girder crossing of the Nickel Plate over the Illinois Central R.R. at 79th St., Chicago, whose design and construction were described in *Engineering News* recently. The structure was painted with three coats of red lead, the second and third coats being tinted with lampblack to different shades. The third, or final, coat was mixed with 6 lb. of lampblack per 100 lb. of red lead, giving a very dark brown color. The total rate of paint consumption on this structure was 1 gal. per 20 tons of steel, but in considering this it must be mentioned that the floor steel is incased in concrete and thus does not expose any painting surface; if the figures are corrected by taking account of the tonnage not painted, the steel covered is about 11 tons per gallon, according to George H. Tinker, Bridge Engineer.

The lower 5 ft. of the Sewall's Point coal pier, Virginian Ry., was painted three coats of red lead—the top coat black and the second coat brown, obtained by different admixtures of lampblack. F. F. Harrington, Engineer of Structures of the railway, says that the following paint mixtures were used, after cleaning the steelwork and removing the rust with scrapers and iron brushes:

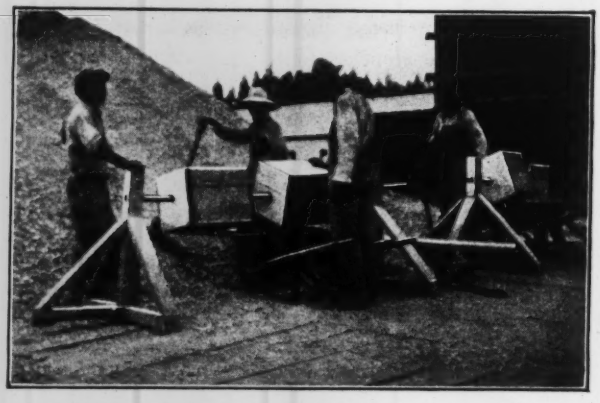
	First Coat	Second Coat	Third Coat
Paste red lead, lb.	100	100	100
Lampblack-in-oil, lb.	...	6	52
Chinese blue-in-oil, lb.	...	...	16
Linseed oil, gal.	2.9	3.64	15.20
Paint, gal.	5.02	6.42	24.55

For general use the National Lead Co., which is largely interested in paste red lead, recommends lampblack additions ranging from a few ounces to about 6 lb. per 100 lb. red lead. For a light brown, 12 oz. of paste lampblack is added; 4 to 5 lb. will give a dark brown, and 6 lb. a very dark color. These browns lack all the glaring orange quality of the natural red-lead color. Handsome green colors are obtained by the use of chrome yellow and Prussian blue; 31 lb. of chrome yellow and 13 lb. of Prussian blue per 100 lb. of red lead give a rich dark green. As this addition is accompanied by a considerable increase of oil, the expensive colors do not add to the cost of the paint as much as might appear. At normal peace-time prices the last-mentioned paint costs about 25c. per gal. more than red-lead paint, according to F. M. Hartley, of the National Lead Co.

Measuring Concrete Aggregate in Revolving Boxes

By H. B. Andrews*

The revolving measuring boxes shown in the accompanying view have been successfully used by the Coronet Phosphate Co., of Hillsborough County, Fla., in the construction of a reinforced-concrete dry bin. The yardage did not warrant the construction of sand and stone bins, and the mixing yard consisted of the usual



CONCRETE AGGREGATE MEASURED IN REVOLVING BOXES

loading platform adjacent to the cement shed. Both the sand and the stone had to be wheeled up runways to this platform, and the revolving boxes were used in measuring both the sand and the stone. The cement was measured and discharged direct into the mixer.

The boxes are proportioned in size to hold the exact quantity of each material and are placed on 1¼-in. pipe as axles, with two or more boxes to the stand as desired. The pipe is carried through holes in the 4x4-in. posts, which are braced and carried on a 4x4-in. support.

A dry batch of 10½ cu.ft. of 1:2:4 mixture was used and required three 2-cu.ft. stone boxes, two 1½-cu.ft. sand boxes and one 1½-cu.ft. cement box. The framework was light and could easily be moved into the stone or sand piles or from one job to another. The whole framework is detachable and can be taken down in a very few minutes.

The bottoms of the boxes were high enough to allow a wheelbarrow beneath in such a way that the box could be turned on the pipe, discharging the load and emptying itself for loading. The apparent objection to this form of loading is in the height of shoveling the material, but by using scoops the boxes were quickly loaded and the additional height was not objectionable. Delays in loading were eliminated, and a uniform mixture was obtained.

Bedding Cement Pipe Sewers in Lean Concrete

Experience in Austin, Tex., with concrete sewer pipe made by city day labor at the city plant has been quite satisfactory. The concrete is mixed 1:3, using a natural gravel aggregate from the Colorado River bed. It is mixed very dry and thoroughly rammed into steel forms, so that the forms may be removed a short time after the pipe is cast. When removed from the forms the concrete has about the stiffness of green clay sewer pipe. The pipe is kept moist and cured in the shade. The concrete is mixed with a small batch mixer.

The only weakness developed in this pipe and the only fractures found in storm sewers constructed of it have been traced to improper bedding of the invert. The soil under the city streets is a hard lumpy gravel, and it is not possible to secure a good bedding by tamping it around the pipe. This problem has been solved by the use of 1:18 concrete for bedding the lower half of the pipe.

The 2-ft. sections of pipe have plain ends, without bells or spigots, and the joints are made by butting the ends together and plastering them inside and outside with 1:1 mortar. Ordinary curves and bends are made by cutting away the pipe and fitting the ends as close together as possible and then filling the joints with mortar. The omission of bells not only represents a saving in cost of construction, but prevents loss from breakage in handling.

Spacing of Shear Steel in Concrete Beams

By Robert S. Beard*

The location of stirrups in a reinforced-concrete beam is generally graphically determined as shown in the accompanying diagram, which is a method of subdividing the shear triangle into n equal areas to be provided for by n bent-up rods or stirrups. First, a semicircle is drawn on AB, the length leg of the shear triangle. Next, this leg AB is divided into n equal parts, and perpendiculars are erected at the division points. The compass is then centered in the vertex A, and concentric arcs of circles are drawn passing through the intersection of each perpendicular with the circumference of the semicircle. Perpendiculars to the leg AB drawn from its intersections with these arcs divide the triangle into n equal areas. The rods are then spaced in the beam so as to pass through the intersections of the neutral axis with the perpendiculars to AB drawn through the centers of gravity of the n respective areas.

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TABLE SHOWING SHEAR STEEL SPACING IN REINFORCED-CONCRETE BEAMS

The figures give distances from supports in decimal parts of the length of the shear triangle

	Number of Shear Bars in Half of Beam																			
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
0.3334	0.5286	0.6151	0.6667	0.7018	0.7278	0.7480	0.7643	0.7778	0.7892	0.7991	0.8075	0.8151	0.8218	0.8279	0.8333	0.8383	0.8429	0.8471	0.8509
2	0.1381	0.2962	0.3905	0.4548	0.5023	0.5392	0.5690	0.5937	0.6146	0.6327	0.6480	0.6619	0.6742	0.6853	0.6952	0.7044	0.7128	0.7204	0.7274
	3	0.0886	0.2107	0.2939	0.3555	0.4033	0.4419	0.4739	0.5009	0.5243	0.5442	0.5622	0.5781	0.5925	0.6053	0.6171	0.6280	0.6380	0.6470
		4	0.0654	0.1639	0.2368	0.2934	0.3391	0.3770	0.4090	0.4367	0.4603	0.4816	0.5004	0.5175	0.5326	0.5466	0.5595	0.5713	0.5820
			5	0.0516	0.1343	0.1986	0.2504	0.2933	0.3296	0.3611	0.3878	0.4120	0.4333	0.4527	0.4698	0.4858	0.5004	0.5137	0.5258
				6	0.0428	0.1138	0.1711	0.2186	0.2587	0.2935	0.3231	0.3498	0.3734	0.3948	0.4138	0.4314	0.4475	0.4623	0.4757
					7	0.0365	0.0988	0.1504	0.1940	0.2319	0.2640	0.2931	0.3187	0.3420	0.3626	0.3818	0.3993	0.4154	0.4299
						8	0.0320	0.0874	0.1342	0.1749	0.2094	0.2409	0.2681	0.2932	0.3153	0.3359	0.3548	0.3720	0.3876
							9	0.0284	0.0783	0.1215	0.1583	0.1915	0.2208	0.2475	0.2711	0.2930	0.3131	0.3314	0.3480
								10	0.0255	0.0713	0.1101	0.1452	0.1762	0.2044	0.2294	0.2525	0.2738	0.2932	0.3107
									11	0.0236	0.0645	0.1014	0.1340	0.1636	0.1898	0.2142	0.2365	0.2569	0.2754
										12	0.0208	0.0594	0.0935	0.1245	0.1520	0.1774	0.2009	0.2222	0.2415
											13	0.0197	0.0552	0.0876	0.1162	0.1427	0.1671	0.1894	0.2095
												14	0.0179	0.0515	0.0813	0.1089	0.1342	0.1574	0.1783
													15	0.0170	0.0478	0.0764	0.1027	0.1266	0.1483
														16	0.0156	0.0452	0.0723	0.0972	0.1196
															17	0.0151	0.0431	0.0687	0.0918
																18	0.0142	0.0406	0.0644
																	19	0.0135	0.0380
																		20	0.0124

Bar Number Counting from Center of Beam

The subdivision of the shear triangle can also be done analytically and the following formula arrived at:

$$z = a\left[1 - \frac{2}{3\sqrt{n}}(s^{\frac{3}{2}} - s - 1^{\frac{1}{2}})\right]$$

in which

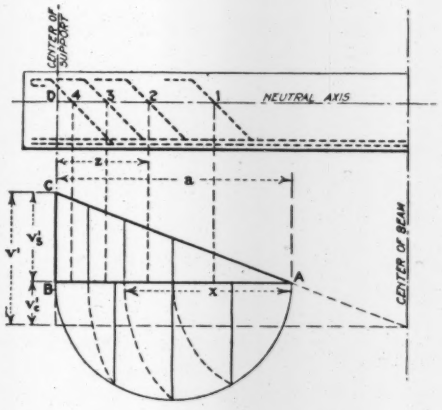
z = Distance of center of gravity of any subdivision from B;

a = Length leg of shear triangle;

n = Number of subdivisions;

s = Subdivision or rod number counting from A.

The length a is the only value in this expression which for any given total number of rods is dependent upon the particular shear triangle in question. The accompanying



table, which has been calculated by Clifton M. Kerr, of the Kansas City Engineering Department, and the writer, is a series of solutions of the quantity within the brackets.

If the base of the shear triangle is drawn parallel to the neutral axis of the beam, the total number of stirrups n needed in one-half of the beam can be found by dividing the product of the area of the steel shear triangle and the breadth of the beam by the working strength of one stirrup; that is,

$$n = \frac{v_s' ab}{P} \qquad (a)$$

but for bent-up rods, as explained hereafter,

$$n = \frac{v_s' ab}{1.42P} \qquad (b)$$

where

v_s' = Part of the shear carried by the steel;

b = Width of beam;

P = Working strength of one stirrup.

The distance along the neutral axis of each stirrup or rod from the support is then found by setting the value of a on the slide rule and multiplying it successively by the coefficients in the proper n column in the accompanying table corresponding to the different rod numbers.

The analytical method of determining the proper spacing of the web reinforcement is made more rapid than the graphical method by the use of this table.

The strength P required of a stirrup or rod is $P = vbs$ for the stirrup and $P = vbs \cos 45° = 0.707\ vbs$ for the bent-up rod. From this it is evident that the value of s for a bent-up rod may be 1.42 times that of a stirrup of the same cross-sectional area. Therefore, a smaller number of bent-up rods than stirrups of the same strength are needed to provide for any given shear area. Of course, the longitudinal spacing of the stirrups or bent-up bars should not exceed three-fourths of the depth of the beam in any case.

The companion table to this, showing where rods are no longer needed for moment, was published by M. J. Lorente in *Engineering News*, Mar. 20, 1913, p. 571.

Location Survey Methods for Irrigation Canals

The following is a detailed description of the procedure in locating the Main Canal, Division 4, from Sta. 1933 west, Grand Valley Project, United States Reclamation Service. This work was done in 1915 by Lyman E. Bishop, locating engineer, now Deputy State Engineer of Colorado, and C. H. Howell, surveyman, Reclamation Service. These surveys and those for location of the laterals of the Second District, subsequently described, were made under the supervision of S. O. Harper, Assistant Engineer, in charge of all fieldwork on the project. The Project Manager is J. H. Miner.

For the main canal, which was to carry from 330 to 120 sec.-ft., the "fly line" method of location was used throughout. A diagram for each canal cross-section was prepared, which showed excavation and embankment quantities (these latter increased 10% for shrinkage) for all center-line cuttings to be encountered. The diagram was so arranged as to show the "economic" center cut required to produce a balance between excavation and embankment for ground slopes from 0 to 50%.

The "fly line," first established, is essentially a grade contour, marked at each station by a lath stake. These stakes were set by level at the proper economic cut for the existing ground slope. The location to fit the topographic conditions was then worked out in the field. All

curves were given a radius greater than six times the bottom width of the canal. Ridge cuts and fill locations were adopted only after various alternatives had been run out on the ground and the comparative yardage estimates had demonstrated the economy of these ridge cuts or fills. As soon as the location line was established, levels for a profile were taken on the center line. Transit and level notes were kept in the usual manner, but in addition to the ordinary level notes a record was kept of the ground slopes, to be used in the estimates.

The office work included the preparation of a profile and alignment map, yardage estimates and a haul diagram prepared from the yardage estimates. Topography taken by transit and stadia at the more important crossings was plotted to accompany the alignment map.

The location party consisted of ten men as follows: A locating engineer, a transitman, a levelman, two chainmen, a rodman, a draftsman, a back-flagman and two teamsters (with their teams). In less than two months this party completed the location of 22.1 mi. of canal, involving the running of about 25 mi. of transit lines and 46 mi. of levels. An average progress of 0.42 mi. per day was attained. Some few changes were subsequently made in this line by C. H. Howell, who was in immediate charge of the surveys for location of laterals.

Methods Used in Lateral Location

Subdivision surveys, based on the section and quarter-section corners of the public-land surveys, were generally the first to be made. The approximate location of the laterals was roughly projected on a contour map and this was taken into the field and studied during the subdivision survey. The acreage was obtained from this map for each system of laterals and a ditch cross-section chosen (from a previously prepared table) which would be large enough to meet the requirements. All laterals were designed to deliver 1 sec.-ft. to 40 acres.

Three general methods of location were used, depending on the topography of the area in question and the proximity to the main canal—(1) the contour method, (2) the land-line method, and (3) the ridge-line method.

1. The contour method was used on all sidehill work and in paralleling the main canal on a grade contour. From the grade at the point of departure at the main canal a line of lath stakes was set by level, rod and chain, according to the economic cut and on the rate of grade adopted. The economic cuts for the different side slopes were obtained from a table, previously prepared, as noted before.

This line of stakes was then lined in so as to reduce the P. I.'s to a minimum, and the external distances were measured to determine the best alignment. A transit line was then run from P. I. to P. I., and if the lateral was large enough to warrant, curves were fitted; otherwise the angles were simply read and ties made at land-line crossings. Levels were then run over the located center line for a profile and the slopes taken at intervals for the preliminary estimates.

2. The land-line method was adopted wherever it was considered at all feasible. This consisted of locating the center line of the laterals on the boundary lines of the 40-acre tracts or on 30-ft. offsets from the section and quarter-section lines. Levels were run over the lines thus established, and subsequently a satisfactory grade line was laid out.

3. The ridge-line method was used where it was found impossible to make deliveries from a land-line location. This is the simplest method of locating ditches, and consists merely in determining the high ground of the ridge and following it down with the best alignment possible. Profile levels were then run and a satisfactory grade line established.

A full location party consisted of eight men—a locating engineer, a transitman, a levelman, a rodman, two chainmen, a stakeman and a flagman. Often a smaller party was used to advantage. In about ten months' fieldwork 53.2 mi. of laterals were located and about 21 mi. additional were surveyed in the course of the investigation work.

The foregoing information is taken largely from the office records of the Grand Valley Project at Grand Junction, Colo.

Shooting Out Tunnel Heading Under 150 Ft. of Water

What is thought to be the first instance of blowing out the final rock heading between a completed hydraulic tunnel (lined and fitted with control gates) and the body of water which it tapped, so that the tunnel immediately went into service, was reported when on Feb. 14 the face of the Alaska Gastineau Mining Co.'s Annex Creek tunnell was blown out into Annex Lake. Great interest attaches to the work, since the depth of water on the face was about 150 ft.

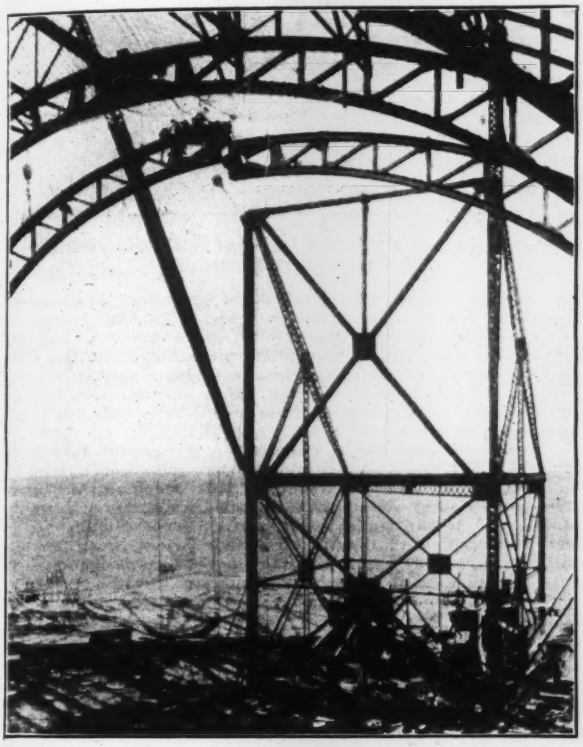
Several months before the final work, diamond-drill holes had measured the thickness of rock to the lake. The face was carried to within 5 ft. of the water, and a pocket was cut on each side of the tunnel. From these pockets final measuring holes were driven top and bottom, using regular machine drills (Sullivan water type) working through a valved nipple. Then 84 holes were drilled 8 in. apart in the face, with machine drills working to within 12 in. of the water. A few holes were very wet so that only 75 were loaded. About 750 lb. of 40% blasting gelatin was used. The temperature ranged about 20° below zero, but the gelatin did not freeze. Fire clay was used for tamping—expensive material, but the only clay available. Waterproof caps were not kept in stock so that electric fuses were waterproofed by dipping in sealing compound, taping and coating with paraffin paint. Each cap was tested before going in and the circuit was later tested (by galvanometer).

A bulkhead was built up of fine muck about 50 ft. back of the heading where the tunnel made a turn. A second similar barrier was placed about 300 ft. farther down. Then the outlet gates were closed and water was turned in from the outside. The tunnel was 1,400 ft. long, 7 ft. high and run on a 1% grade, so that when the portal was covered the water extended back only 700 ft. The water was allowed to rise until it reached the bulkhead nearest the portal—about 1,050 ft. in. The shot was fired with a No. 5 blasting machine. A pressure gage on the water in the tunnel registered only 10 lb. rise, although the concussion shook the snow off the trees for several hundred feet around. The waste and head gates were then opened and the tunnel allowed to clear. Then water was turned into the power pipe line. The work was done under H. L. Wallenberg, Chief Engineer, and J. A. Wilcox, Assistant Chief Engineer of the company.

Erecting Steel Arch Trusses

The erection of the steel-frame buildings on the new Municipal Pier at Chicago included the placing of large arched trusses for the main hall of the recreation building. This hall is 140x150 ft., and is to be used for public concerts and other entertainments. The pier and its superstructure were described in *Engineering News*, July 29, 1915.

The main framing of the great hall consists of three-hinged steel arch trusses having a span of 136 ft. 8 in. c. to c. of shoe pins and a rise of 74 ft. 4 in. from shoe



ERECTING ARCH TRUSSES FOR THE RECREATION HALL ON THE CHICAGO MUNICIPAL PIER

pins to crown pin. There are four of these trusses, spaced 20 ft. c. to c., connected by longitudinal girders and purlins. The end of the hall is semicircular, and has ten half-trusses placed radially. These have the two panels omitted and their heads are attached to a semicircular ring girder carried by the last of the full-arch trusses. To relieve the pressure upon this girder, the backs of the radial trusses have cantilever framing on the lower portion, which supports a part of the superstructure and thus acts as a counterweight to the half arch.

The chords are pairs of channels braced together and having cover-plates on the lower portions. The web members are built up of angles. The hinge pins are 4 in. for the shoes and 3½ in. for the crown. Above the trusses is a monitor roof.

For the erection, a steel-frame traveler was used, mounted on wood rollers on two runways about 36 ft. apart. The traveler consisted of a rectangular tower 38½ ft. square and 32 ft. high (or 37½ ft. above the ground), carrying at the top of its forward frame the two 40 ft. masts with back braces to the rear frame. Each mast had a 90-ft. latticed boom. The masts were connected by a top strut and bracing, as shown. The traveler carried two hoisting engines, one with three drums and the other with two drums. Its weight complete was 50 tons, and the hoisting capacity for each boom was 10 tons.

The steel was sent by freight on the Baltimore & Ohio R.R. to 12th St., where it was unloaded by a gantry yard crane and hauled by team to the site. The largest member handled weighed about 3½ tons (the crown segment of the arch) and was about 6½ ft. and 38 ft. long.

The arches were assembled complete on the ground, well bolted up, in two segments. Each segment, forming a half arch, was raised by one boom. The chain was attached at about 37 ft. from the head of the truss, the weight of this overhanging portion being about 3½ tons. The two booms were operated simultaneously and raised the two segments until the top ends came into position. The crown pin was then driven. The lower ends of the segments rested on the cast-steel shoes while the lifting was being done, so that each boom was holding only part of the weight of the segment it was lifting. The weight of the half arch was 10 tons, and each radial arch weighed 9 tons. The first full arch was first erected, and then the traveler worked backward to the outer end of the pier.

The total amount of steel placed was about 1,150 tons, and this was erected in the period from Aug. 15 to Oct. 15, 1915, with an erecting force which averaged about 40 men. No shop painting was done, but two coats of red lead were applied in the field.

An interesting feature of the work was the use of manila nets to protect the erectors. These were 12x40 ft., attached to the steel framing below points at which the men were at work. This use of protective nets, as required by the State Factory Inspection Department, was noted in *Engineering News*, Mar. 2, 1916.

The steel framework was built and erected by the Morava Construction Co., of Chicago. The pier and buildings were designed and built under the direction of the Harbor and Subway Commission: E. C. Shankland, Chairman, and W. L. Artingstall, Harbor Engineer.

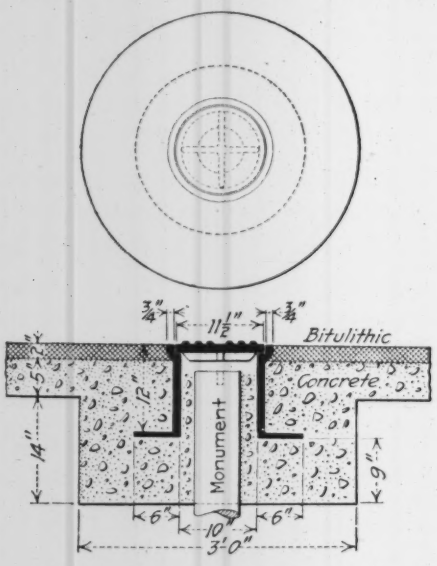
NOTES

Pulling Up Stumps and Trees with Gasoline Tractors is practiced extensively in the Southern States. With old stumps whose deep roots have rotted, a direct pull by the engine may suffice; but if the cable is led over a pulley on top of a tripod, it gives a lifting effect that helps to pull out the roots. The same effect is given by what is called a "dutchman." This is a strong fence post about 5 ft. long, driven into the ground at an angle of about 30°, with its upper end over the stump. The chain or cable wrapped around the stump is led over a notch in the top of this post and thence to the tractor. The strain on the cable tends to raise the post, and the leverage thus gives a vertical lift. For large green stumps the tractor may be used to operate a stump-pulling device.

Water-Tight Sewer Construction is practically impossible in quicksand, water-bearing sand or soft muck under ordinary conditions and with the usual materials, according to a paper read before the Iowa Engineering Society by Charles P. Chase, of Clinton, Iowa. If the cement is covered, the water bores it out; if it is left to harden before covering, the weight of the shifting sand or the movement of the pipe cracks the joint open. In a job in wet sand Mr. Chase had good results by using a 1:1 cement-mortar (made as stiff as it could be worked) on top of well-calked oakum joints and then plastering the joint with a waterproof putty-like blue clay found on the job. To overcome the objections to cement, he has used sulphur compounds, tar compounds and asphalt, but

not with much success. The sulphur compounds frequently became too hard and cracked open and were inferior to cement. The tar and asphalt compounds disintegrated or dissolved in a few years. Some compounds are used hot and poured into the joint, but the calking must be absolutely tight or the jointing material will go through to the pipe. It also requires a tight gasket around the joint and the exclusion of dirt and water when the jointing is poured. Where the trenching is in clay, fairly tight work can be secured if care is used, but the clay must be packed tightly under and around the pipe. With trenching machines that move rapidly Mr. Chase has found it difficult to secure this proper tamping around the pipe, especially when the automatic backfiller is used, but it can be done. The use of subdrains and tile to keep groundwater out of sewers is not successful, as the sewage escapes and pollutes the water in the drains. Ordinarily Mr. Chase would not guarantee a leakage into a sewer less than 10,000 gal. per mi. in ordinary soils, or 20,000 gal. in water-bearing sand or quicksand, and he usually figures on the latter amount.

The Standard City Survey Movement of Pueblo, Colo., is shown in detail in the accompanying sketch. The monuments are placed in the pavement area on the street center



CITY SURVEY MONUMENT USED IN PUEBLO, COLO.

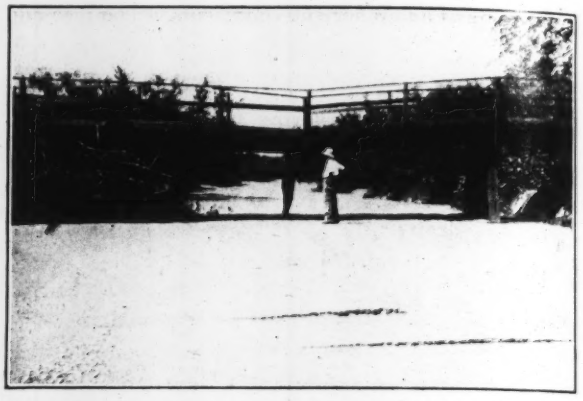
lines or on short offsets from the center line. The city engineer is D. P. Gaymon. The cast-iron box and cover weigh respectively 140 lb. and 15 lb.

The Cost of Track Weeding on 16 electric interurban railways is given in the February "Question Box" of "Aera," the official journal of the American Electric Railway Association. This compilation has been made by Edward Karow, Assistant to the Vice-President, Twin City Rapid Transit Co., Minneapolis, and appeared in reply to a question as to the best method of weeding. The figures of annual cost per mile of single track are as follows:

Company No.	Manual Labor	Chemicals	Weed Burner
1	$37.50	....	$6.25
2	50.00	$23.90	....
3	....	....	7.50
4	....	6.00	....
5	20.00	....	15.00
6	27.04	....	....
7	52.50	....	....
8	....	....	4.66
9	....	36.54	....
10	18.30	....	....
11	58.00	12.00	12.00
12	55.00	25.06	....
13	50.00	43.55	....
14	75.00	25.00	....
15	50.00	33.00	....
16	70.00	28.80	....
Average	$47.00	$26.00	$9.00

Some companies clean their track to the end of the ties and others for a considerable distance beyond. Where manual labor is relied on, the track has to be weeded from one to three times per year; but the track is left in the cleanest condition. Burning is the cheapest scheme; but the track is left unsightly, and the work has to be repeated as often as hand weeding. Chemical treatment is the most effective; the cost is high in the first year, but lower in the second and third years.

A Bridge Defying the Laws of Mechanics by not opposing is shown in the accompanying illustration. It is a creek crossing in Pima County, Arizona. In one of the occasional floods to which these "streams" are subject the center bent

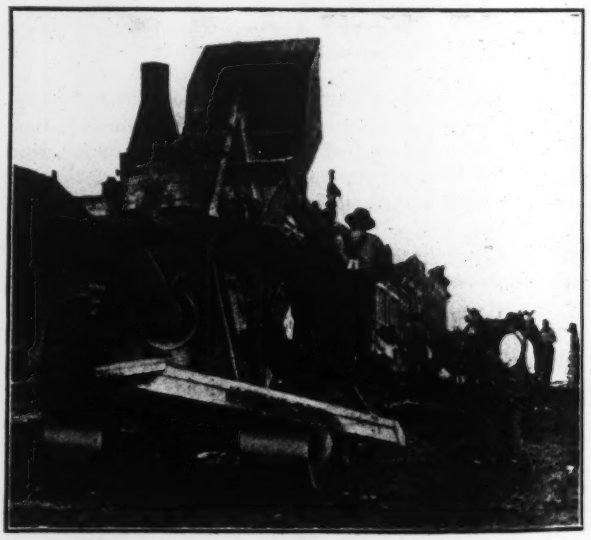


A WASHED-OUT ARIZONA COUNTY BRIDGE WHICH CARRIED AUTOMOBILES

was washed out, and one of the other bents was knocked out of plumb by drift. The bridge is made of 3x14-in. joists laid 24 in. on centers and floored with two courses of 2x12-in. planks laid diagonally. The joists were lapped at the ends about 10 in. and nailed. The bridge in the condition shown, with no center support, carried safely several automobiles and an unknown amount of other traffic before its condition was reported.

Commercial Museum at Calcutta—A permanent exhibition of Indian and foreign manufactures has just been opened in Calcutta, India. The primary object is to encourage the country's industries by showing the Indian manufacturer not only samples of Indian-made goods alongside of similar wares now imported, with details of selling prices, quantity imported, etc., but by exhibiting samples of foreign-made goods with information as to the trade therein. Particular attention has been given to exhibiting samples of competing goods from German, Austria, Japan and the United Kingdom.

A Mixer as a Road Roller was a novel contracting device employed by Samuel Friedman, of Glenwood, Iowa. Steel wheel casings, made of ½-in. metal and 20 in. long by 30 in. in diameter, butt-welded, were slipped on over the wheels



PAVING MIXER USED AS A ROAD ROLLER BY INCASING THE WHEELS

of a Chain Belt paving mixer. The outer half of each tire was filled with a 1:2 mix of cement and sand. Mr. Friedman states that this arrangement cost him $75. It would have cost this much to ship a roller to the job.

Editorials

Elmer L. Corthell

Probably no member of the American engineering profession had a wider acquaintance than Elmer L. Corthell, whose death, on May 16, was announced in our last issue. Yet the most important engineering works on which his claim to professional distinction rested were carried out 25 to 50 years ago. His intimate friends will recall that a score of years or more ago Mr. Corthell had a severe and long-continued illness, which robbed him of the sight of one eye. His restoration to health was very slow, and his retirement from the active practice of his profession may be said to date from that time. Although he received afterward a few notable commissions, such as membership on the Advisory Board of the New York Barge Canal, his days as an active force in the construction world were over.

Most engineers who step out or are crowded out from professional work after they have achieved important positions and distinction find that the busy world soon forgets their existence, save a few old-time personal friends; but Mr. Corthell was without doubt far more widely known at his death than when at the summit of his active professional career as a construction engineer he built the West Shore R.R. in 1884.

His recent election to the Presidency of the American Society of Civil Engineers was of course an important reason for this; but that office, like the offices he held in other engineering societies, came to him because of the services he had rendered to the profession in connection with its organizations, and the distinction he had gained thereby.

All his life long, but especially when he found himself with spare time on his hands after his long illness, Mr. Corthell was ready to devote himself to the work of engineering organizations, with the single-hearted purpose of benefiting the profession. At the time he gave his library to Brown University, a few years ago, Mr. Corthell was a member of 25 engineering societies and congresses, American, foreign, and international. And membership with him invariably meant work. He did not become a member of a society to see what he could get out of it for himself, but to give to it whatever service he was able in its work for the profession.

Particularly was he interested in the international congresses of engineers which have played such a great part in the past twenty years in promoting the solidarity of the profession, although now unfortunately interrupted by war. Mr. Corthell's prominence in the International Railway Congress, the International Navigation Congress, the Pan-American Scientific, and others, with his long residence and extensive work in foreign lands, made his name better known abroad probably than that of any American civil engineer of the present day except General Goethals. The profession needs more men like Corthell, willing to devote spare energies in their declining years to work for the general welfare. They will find, as he found, that such work is well worth the doing.

Selecting Hydraulic Turbines

Great advance in American turbine and hydraulic power-station designing has been made since engineers on this side of the Atlantic began to use the concepts of "type characteristic" and related ideas. The peculiar advantage of the "type characteristic" as a criterion of turbine design lies in its simultaneously measuring the difference between various designs of runners, and combining the controlling hydraulic conditions (speed, power and head) of a given site into a single factor which indicates what sort of a wheel is best suited. The development of efficiency-contour diagrams (iso-efficiency lines on a field having speed and power coördinates), which have come into use in this country along with the idea of type characteristic, has been a second great aid in promoting the study of turbine runners by showing on a single sheet, as it does, the efficiencies which can be secured from a given machine at all speeds and outputs within its working range.

It has remained for Messrs. Horton and Pomeroy (as is shown elsewhere in this issue) to connect up type-characteristic and efficiency-power-speed studies, and thereby to simplify and promote to a much greater extent than heretofore possible the studies of turbine design which must precede an intelligent selection of equipment. They have approached this desirable end first by making the speed and power coördinates for their isograms proportionate quantities rather than absolute—referring as unity to speed and power for maximum efficiency. Now by mentally reducing the dimensions of the runner under study until the absolute value of power becomes unity under 1-ft. head, the corresponding best-efficiency speed is the type characteristic. This makes the proportionate-scale isogram apply to all the homologous runners having this type characteristic—it enables making almost at a glance the fundamental comparison (neglecting of course those inevitable minor differences between homologous runners of different and even of the same size). By introducing two constants, one the diameter of the hypothetical type runner which develops 1 hp. under 1-ft. head, and the other the product of diameter and speed of this runner, it is possible quickly to compute the diameter of a given type of runner which will give the required capacity and speed at best efficiency.

The authors have naturally confined themselves solely to a discussion of their system of turbine study, but attention should be called to the tests of one of the turbine runners, cited by them as an example for applying the new studies. The runner designated as "Holyoke Test 2267" was designed by Mr. Pomeroy in an effort to secure for industrial mills a machine of maximum flexibility—a wide range of speed at nearly constant efficiency, and high efficiency with constant speeds over a wide range of head. Knowing that the ways in which these ends are reached are generally considered incompatible with securing the very highest attainable efficiency, the

performance of this runner is really remarkable. Its maximum efficiency is 88.86%, under 25-ft. head and 220 r.p.m., a figure which compares with the best record of any type of turbine—93.07% for the Leffel runner designated as Holyoke Test 2359. For 20-ft. head the constant-speed efficiency is 87.3%; for 36-ft. head it is 85%. For full gate the capacity runs from 210 to 515 hp. So far as has yet been demonstrated, this design gives the highest efficiencies over the widest range of any runner yet built.

Aside from the obvious economy of having such a runner in a one-unit mill where the load is not constant (and there are a few engineers for hydro-electric operating companies who believe that well sustained efficiency is always more important than a high peak no matter how good the load factor) such a flexible design is well adapted for service where overload capacity is needed. Few men and too few engineers realize that a hydraulic turbine can have overload capacity—thinking perhaps that a good turbine necessarily has to be operated at full gate for normal power. "Full-gate" today is somewhat of an arbitrary position and there can be a wider opening than required for maximum efficiency and normal capacity, the added power at higher openings being the overload capacity. Up to now serious computations of performance of a turbine selected for large overload capacity have been attempted only by a few specialists. Now power to do this is at any engineer's disposal, as illustrated in the Horton-Pomeroy article, and better still overload performance is visualized in the diagrams presented.

Drastic Sewage-Disposal Law

Texas would be a rich field for engineers, contractors and supplymen this year if its State Board of Health were to undertake and succeed in the literal enforcement of the drastic anti-stream-pollution law printed elsewhere in this issue. The sweeping provisions of the law apparently contemplate that all sewage in Texas, except on tidal streams, must be treated to a high degree by Jan. 1, 1917. Failure to see that this is done renders individuals, directors of corporations and members of municipal governing bodies liable to cumulative fines, injunction and, if the injunction be not obeyed, then to punishment for contempt of court.

On the face of the statute there is little chance to escape from the mandatory provisions of the law to cease polluting water used for "farm, live-stock, drinking and domestic purposes." The only hope of warding off the necessity of treating all sewage so as to bring it to a drinking-water standard is (1) the chance for litigation over the meaning of "pollution" and over the reasonableness of the act and (2) that the enforcement of the law is left to the State Board of Health, which obviously must exercise discretion in its efforts at enforcement—all the more so because the law provides for only a single inspector of pollution. There is evidence that the board and its sanitary engineer are going slow in the enforcement of the law, but it seems to be a serious error to pass legislation so drastic that those responsible for its enforcement virtually have to suspend it entirely as regards many cities and largely as regards all.

Since so much discretion must be used in violation of the law, how much better it would have been to have given the board explicit power to compel the erection of sewage-treatment works wherever their necessity is dictated by the interests of public health or the prevention of public nuisance, the character of each plant and its efficient operation to be within the control of the board. At the same time the board—if it now lacks it, as is likely—should have been charged with the far more important power and duty of seeing that every municipality or company in the state supplying water furnishes nothing but water that is hygienically safe.

Doubtless many Texas cities will, spurred on by the law in question, establish some sort of sewage-treatment works—and operate them shiftlessly—when the same money might be better spent for some public works or service having a far more direct bearing upon public health.

The Texas Stream-Pollution Act is commendable in that it shows a desire to protect the public health. It is unfortunate through betraying an ignorance of the fundamentals of sanitary science and sound public policy—and of knowing how to make a dollar contribute most to the protection of life and health.

Light on Flat-Slab and Other Concrete Floor Tests

While on the face of the returns the most important feature of the extensive concrete floor tests at Seattle, noted on another page, is the superior residual strength of the girder and slab floor as compared with the flat-slab, or at least a greater margin of safety in the former under current design methods, one should not overlook the less obtrusive but equally important excuses for the potential failure of the mushroom floor.

One of these excuses, the claim that the slab did not set up on account of cold weather, is not sufficiently answered in the report. The four-way slab was poured in December and tested in April, and the mean temperature of the period was about 45° F. The other two floors were made and tested in warm weather. Concrete 113 days old may reach a fair proportion of its final strength under such conditions, but the burden of proof is on those who claim that it does and in the report the testers show no figures proving the integrity of the concrete itself. Concrete cylinders cut from the floors approximate the strength of those cast separately, but data as to the time of cutting or manner of storage in curing are not given.

In this first excuse the designer of the floor has the better of the argument, unless more is known of the concrete than appears in the report. Had such a building collapsed, can one doubt that the investigators would have brought up the time-worn cry of frozen concrete?

The second excuse is that the steel was grievously misplaced in construction, and unexpected loads thereby thrown on the steel and concrete. This is a most unfortunate argument to be made by one engaged in the commercial promotion of concrete buildings. If this system of design is so delicate that construction errors made by skilled if not expert workmen, foremen, and engineers, honestly endeavoring to follow design, can cut in half a computed strength, the design is obviously not proper for engineering construction.

Entirely aside from these criticisms, the Seattle tests contain much that should interest the Joint Committee

which is now laboring on the flat-slab problem. The four-way test confirms some generally held doubts as to certain mathematical gymnastics; the two-way test, while certifying the main points of its design, emphasizes the dangers of wall column eccentricity, and the beam-and-girder test, as Mr. Hooker so well points out, shows how much of the strength of such structures is due to factors not considered in design.

Higher Pay and Shorter Hours

For many years the eight-hour day has been the slogan of leaders in the labor movement. They succeeded years ago in establishing the eight-hour day in nearly all Federal Government, state, and municipal work, but, up to a year ago, they have had little success in securing the adoption of the eight-hour day in industries carried on by private enterprise. The enormous demand for labor during the past year, however, and the cutting off of the supply of immigrant labor from Europe have given the labor leaders their long desired opportunity. More progress has been made in securing the eight-hour day in the past year than in the preceding quarter century.

In the machinery trade, it is stated by a writer in the *Survey*, that whereas on Jan. 1, 1915, only 7,000 members of the International Association of Machinists were working eight hours, on Jan. 1, 1916, no less than 60,000 were working eight hours. In the shops of Springfield and in Bridgeport, two of the most important centers of machinery manufacture in New England, the eight-hour day has been adopted during the past year. If it continues there, Worcester and Providence will have to follow the example or lose many of their best workmen.

In the anthracite mining industry an agreement reached on Apr. 29, between the leaders of the United Mine Workers and the coal operators, establishes the eight-hour day for 110,000 workers in the anthracite coal fields, with an increase in wages amounting, according to the claims of President White of the United Mine Workers, to at least $10,000,000 per annum.

The burden of this wage increase to the miners is to be borne by the consumers, with something added, apparently, for good measure! The Philadelphia & Reading Coal and Iron Co. on May 9 announced advances in the wholesale prices of coal amounting to 40c. per ton for stove, 50c. for pea, 15c. for egg, 20c. for chestnut, and 20c. for buckwheat. If President White's claim is correct that the advance in wages in the anthracite fields will amount to $10,000,000 and the annual production of anthracite is assumed at 80,000,000 tons, it would appear that an average increase of 12½c. per ton in the selling price of coal would be a sufficient advance to transfer the wage increase to the consumer. Coincident with the announcement of the advancement in price, there came a statement that the Attorney General of the United States has requested the Fedral Trade Commission to make an investigation and determine whether the increase in coal prices is warranted.

It should be noted that by no means all the efforts to secure the eight-hour day are successful. A notable illustration to the contrary is the recent strike in the Westinghouse Electric Co., where the employees returned to work on May 8 as a result of the announcement by President Herr that all those participating further in the strike would be excluded from the company's pension and benefit system. A factor in the ending of the strike was doubtless the very high wages which many of the piece workers on munition production have been making, and which they were not anxious to reduce by the adoption of the eight-hour day.

The most important of all the demands for an eight-hour day is that made by the railway trainmen of the United States numbering over 400,000. Their demand is that the daily wage rate shall remain as at present and that time and a half shall be paid for all time over eight hours. Representatives of the trainmen's brotherhoods and of the railway companies are soon to meet in joint conference in New York City and endeavor to reach a basis of agreement. The *Commercial and Financial Chronicle* of New York, in a recent review of the situation, declares that the railway companies by yielding to the employees' demand that the railways shall deal with the employees collectively, instead of a company dealing with its own employees, have practically surrendered to the trainmen in advance. It says:

> The railroads have delivered themselves bound hand and foot into the control of these labor organizations. Disagreement now means inevitably a general strike and a complete suspension of transportation facilities. That would be a catastrophe of such frightful dimensions and such paralyzing effect that the railroads could not afford to let it occur and the public would most decidedly not allow it to occur.

An estimate has been published that the cost to the railways of the United States of granting the demands of the trainmen will be $100,000,000 a year. There are some 300,000 employees in freight train service who would be affected by such a change to the extent of adding 50% to their hourly wages every day after they had worked eight hours. It is obvious that without a wholesale change in division points and length of runs, which it would take years to accomplish, it will be impossible for the railways to materially change the length of working day of freight trainmen. In 1914 the average daily earnings of enginemen were $5.27; of firemen, $3.24; of conductors, $4.49; of other trainmen, $3.10. A rough estimate based on these figures shows that the estimate of $100,000,000 above noted is not extravagant.

It will be impossible to grant such high rates of compensation and short working days for trainmen without doing something for other classes of railway employees whose wages are at present much below those paid to trainmen for equal service and responsibility. The average wage of railway trackmen for example is only $1.59, taking the average of the whole country, and section foremen receive only $2.21. Within the past year the demand for ordinary labor for work on contract jobs, on farms, in mines and in factories have compelled the railways in many parts of the country to raise the wages of section men in order to hold their forces.

It is of interest in this connection to note that the entire net earnings of all the railways of the United States in 1914 available for the payment of dividends and for improvements and extensions was only $287,000,000 in round numbers. The payment of $100,000,000 in increased wages to trainmen, therefore, would require over one-third of the present profits of the railway companies. If such a reduction in railway earnings takes place, it will most seriously handicap the railway companies in obtaining new capital. On the other hand the congestion of traffic of the past few months has demon-

strated how essential it is that railway credit shall be maintained so that the railways may be able to make the further large investments necessary to increase their capacity. The probabilities are, therefore, that the trainmen's demands will be granted and the public will have to pay the bill in an increase in railway rates.

There is no room for doubt that public opinion holds a very different attitude toward the eight-hour work day from that which was held a quarter-century ago. One finds many men nowadays not alone among workmen and their leaders, but among the leaders in professional and business life and those directly in charge of labor who declare that under modern factory production with high speed machinery and its requirements, 8 hours' work in 24 is as long a working day as the average man can endure without early breakdown. Many firms engaged in industries where profits are large have voluntarily established the eight-hour day. Examples are the Ford Motor Co., the Remington Typewriter Co., the Victor Talking Machine Co. On the other hand, in a strictly competitive industry it is practically impossible for one firm to establish the eight-hour day if competing concerns equally well equipped and well managed are working ten hours. The establishment of the eight-hour day, therefore, in such industries is dependent on the existence of a labor union strong enough to control operations in the entire industry, so that all competing firms have to purchase their labor on the same basis.

There are other industries, however, where the work is dependent on the weather or the seasons, in which no defense of the eight-hour day can be made and no demand exists for its establishment. The farmer must work long hours when it is time to plant and gather his crops, or suffer heavy loss; and he can take his rest at slack seasons and on rainy days. There is a great deal of construction work, such as spreading bituminous oils in road construction, for example, which can be done only in favorable weather, where the limitations of an eight-hour day may constitute a real hardship to the industry and to the workers.

That the general extension of the eight-hour work day will have far reaching effects goes without saying. The first and most obvious effect is an increase in the cost of living. Higher prices for coal, higher rates for transportation, higher taxes because of the eight-hour workers in government employ are typical of the result in all industries. Of course, in any industry where work is so intense that a man can produce as much in an eight-hour day as in a ten-hour day there will be no increase in production costs.

The real test of the eight-hour day will come when it meets international competition. The United States must hold its foreign markets to keep its factories busy and its workmen employed. Sooner or later it must sell its goods again in competition with the factories of Europe. The puzzle is to know what condition these foreign competitors will be in after the war. Were they as well equipped as they were two years ago, the United States would have little chance in the competition except in certain lines where it has great natural advantage.

Certain conditions will handicap the foreign manufacturer after the war. Labor will be scarce and production of all sorts will be under the heavy handicap of enormous taxation made necessary by the war debts. On the other hand, the purchasing power of Europe, as a result of war's impoverishment, will inevitably be greatly impaired and the surplus production available for export may soon be as great as before the war.

According to the best opinion of economists we cannot long maintain the artificially high scale of prices for materials and for labor which now prevail because Europe is borrowing money to spend like water. A merchant may prosper amazingly from the custom of patrons who are spending borrowed money; but when they stop borrowing and become his competitors there is a different story. The high wages and short hours established in 1916 may not all be permanent.

Levee-Building Machines Show Up Well in Test

Nearly all of the Mississippi River levees are simple earth embankments built with mules and scrapers with methods hardly improved in the thirty-odd years of such work. Figures of cost have necessarily varied with time and location, but a fair average is that on the Lower Yazoo district, where in 30 years some 26 million cubic yards of levees were placed at an average cost of 18c. per cu.yd. In times of emergency, as noted by Major Slattery on another page, this cost was apt to jump to 25 to 35c. per yd. Under such conditions it was but natural that those in charge of the Mississippi work should turn to mechanical devices, especially as the field of earth moving has developed so remarkably in the past few years.

In Major Slattery's article the first results of this move toward modern methods are outlined. The results are most satisfactory and the prospects of developing efficient machines most encouraging. Costs as low as 8c. per cu.yd. have been made and the prospective average seems to be from 8 to 14c. per yd. If the Humphreys bill, which passed the House last week with a total appropriation of $45,000,000 (plus a half of that sum imposed upon the States and levee districts), passes the Senate and receives the President's approval the building and enlarging of levees will increase beyond all previous expectations. The added protection made possible by the lowered cost of machine buildings makes imperative the further application of such methods and the discarding of the primitive horse and man labor, particularly if the modern way makes equally good levees.

The world's wisdom is sure to be increased if a bill proposed to the legislature of Massachusetts goes through. The bill reads:

> **Resolved, The Public Service Commission is hereby directed to investigate the relations existing between railroads and railways in this commonwealth; how the facilities of each may be changed, to the end that there may be coöperation between the railroads and railways in the transportation of passengers and freight, and report their findings to the next general court on or before Jan. 15, 1917.**

The relation between "railroad" and "railway" has been a mystery these many years. This much is known: They have been the faithful gold-dust twins of lawyers connected with transportation matters and have coöperated very well indeed, as Tweedledum and Tweedledee. Therefore, the proposed law is quite unnecessary in so far as it seeks to procure coöperation.

Or is it possible that the bill refers to a steam Tweedledum and an electric Tweedledee?

Letters to the Editor

Success in Engineering

Sir—The writers of the various letters in your issue of May 11, in regard to the qualifications for success in engineering, all seem to lose sight of the fact that the object of this inqury, as I have understood it from the statement made by Professor Mann at the last annual meeting of the American Society of Civil Engineers, is to determine the best method of educating engineers, and, while it may be true that the qualifications named are in the order of their importance those most necessary for the successful engineer, it does not seem to me that they are the qualifications which can be expected from the engineering student, or even graduate.

Character and integrity should be taken as a matter of course, but character is not formed without experience. Efficiency, judgment, perspective, understanding of men, and executive ability can only come from experience, and cannot be taught in any engineering school.

I think we are losing sight of the fact that our engineering schools should above all things else ground the students in the *principles* of engineering, and while they should be informed that the other qualifications are necessary to their future success in their profession, they should also have a full comprehension that without a complete and full knowledge of technical principles they cannot hope to succeed. This point of view was pointed out by me to Professor Mann, but apparently had no influence in the composition of the second circular which was sent out. But it does seem to me that this is a point of view which those responsible for the education of engineers should have in mind. F. Lavis.

New York, May 12, 1916.

Getting Grease and Fertilizer from City Sewage

Sir—The experiments on the Miles process of treating sewage by acidulation as carried out at Boston by E. S. Dorr of the Sewer Department since 1911 and more recently by Prof. R. S. Weston of the Massachusetts Institute of Technology, and described in *Engineering News* of May 11, 1916, will no doubt attract the attention of sanitary engineers. The margin between the cost of sludge disposal and the revenue recoverable from its valuable constituents is not likely to be wiped out soon—perhaps never, in the average installation. But it is gradually being narrowed so that, without expecting a net profit, unless in exceptional cases, it is probable that there will be an increasing number of treatment plants where the cost of sludge disposal *may be* appreciably reduced by the separation and sale of the grease and fertilizing constituents. Whether it *will* be is another matter, for, as pointed out by George T. Hammond[1] in his discussion of the activated-sludge problem, the more attractive proposition of garbage utilization is up to the present time neglected in all but 26 of our 199 cities having a population exceeding 30,000. But it is believed that there will be a tendency toward utilization due to advances in the art of sludge treatment and also to the enormously increased demand of late years—even before the war—for nitrates. It has been estimated that the supply of Chile saltpeter—the only supply of any magnitude now known—will be exhausted within 50 years, and although the fixation of atmospheric nitrogen may supplant the natural supplies to a considerable extent, the power required for its production is limited and will tend to appreciate in value as time goes on. For these reasons it is believed that dried sludges high in nitrates or ammonias will become more marketable than in the past.

The grease in many sewages is, however, by far the most valuable ingredient, while at the same time a detriment to the use of sludge as a manure, and in the recovery of this the Miles process appears to be particularly effective. As indicated by Prof. Weston, there may be recovered from sewage containing 51.6 p.p.m., 430 lb. per million gallons by the use of a ton of SO_2, or, according to Mr. Dorr, 393 lb. per million gallons by the use of 1.1 tons of H_2SO_4, at an estimated cost of some $18. This amounts to about 0.215 lb. of grease per pound of SO_2 or 0.178 lb. per pound of H_2SO_4. The market value of the grease and the tankage being about $24, the net profit is estimated at $6 per million gallons of sewage.

The Miles process is similar in many respects to that employed for many years at Bradford, England,[2] except in the source of the grease, which in this case is mostly from wool-scouring, and in its amount, which is 440 p.p.m. or 8½ times that found at Boston. At Bradford 2.75 short tons of H_2SO_4 are used per million U. S. gallons, producing about 2,340 lb.[3] of grease or 0.43 lb. per pound of H_2SO_4, while at Boston there is produced just one half that amount or, as already stated, 0.215 lb. per pound of acid. The difference is probably due to the great difference in the amount of grease in the raw sewage.

The estimated tankage at Boston is 0.68 and at Bradford 3.65 short tons per million U. S. gallons.

The revenue from the sales of grease and tankage at the latter place amounted to about $140,000 in 1912 or $22.80 per million U. S. gallons, which was something less than the gross cost of treatment.

H. W. Clark places the theoretical value of the fertilizing and fatty ingredients of sewage at from $60 to $80 per million gallons[4] and the Metropolitan Sewerage Commission of New York[5] estimated the theoretical value of that city's sewage in 1910 as about $18,600,000, all of which is wasted. In England its value has been estimated by Dr. Grossman of Manchester at over $100,000,-

[1] "Engineering News," Apr. 27, 1916, page 799.

[2] See "Engineering and Contracting," Vol. XL, Nov. 26, 1913, page 601.

[3] Sixteen to 20 long tons per day from 14 million Imperial gallons. "Sewage Disposal," Kershaw, page 150.

[4] "The Fertilizing Value of Sewage and Sewage Sludge," 1914.

[5] Report, 1914, page 349.

000 per annum, and Dr. Gilbert Fowler, until lately also of Manchester, has stated "that there is an almost unlimited outlook for properly prepared sewage manure, either alone or as a basis for the enrichment of artificial fertilizers."[6]

Any method which promises to conserve even a small portion of such values should be encouraged, but past experience has not been such as to create optimism. If the acid process is tried out on a large scale at Boston it will at least furnish information of value to other communities. Its success here, however, would not mean that it would supplant other well-known methods to any great extent, for at Boston the sewage contains a larger amount of grease than the average, and a large volume can be handled from the start, utilizing the ample storage tanks already built at Moon Island. It is also probable that in the yearly operation of a full-sized plant the resulting efficiency would be much lower than in a carefully conducted experiment and this difference might easily wipe out any anticipated profit. In short, it is extremely doubtful if so elaborate a treatment of the sludge would ever be profitable except in dealing with rather large volumes of sewage rich in fatty matters; but where these conditions occur it may be that the Miles process, with its alleged freedom from odor, the sterility of the effluent and the high stability of the sludge, will be shown to have sufficient merit for serious consideration.

Further experience with its use at Boston will be looked for with interest. KENNETH ALLEN,

Engineer of Sewage Disposal, Board of Estimate.
New York City, May 15, 1916.

Montreal Centrifugal Pump

Sir—The editorial entitled "Triple Expansion Still Ahead" in your issue of May 18 might give the impression to the casual reader that the information originally supplied regarding the test of the De Laval steam-turbine-driven centrifugal pump at the Montreal low-level pumping station was intentionally misleading.

I therefore wish to point out that the steam consumption used in figuring the duty of 168.3 million was taken according to the method agreed upon by the City of Montreal, viz., the pumping unit to be charged with the steam actually consumed by it and by the auxiliaries, less an amount of steam equivalent to the heat imparted to the boiler feed water in the primary and secondary heaters.

This is a simple and easily understood basis of comparison and is not "an impossible, hypothetical duty based on utilizing the entire heat in the steam," as asserted by you. In fact, it is the only simple and sensible method for figuring duty where partially expanded steam is used for preheating, an operation which, as has long been known, is an improvement upon the strictly Rankine cycle. GEORGE H. GIBSON.

Tribune Building, New York City, May 19, 1916.

[It was not intended to intimate that the builders of the Montreal unit had issued the statement in question with any idea of gilding the truth. What we objected to, and with reason, is the statement that the duty of a pumping unit is anything more, or less, than that defined by the American Society of Mechanical Engineers' Test Code. What the builders called "duty" required a special warning that it was not duty as understood by water-works engineers. Sooner or later such a qualification of the term was bound to drop off and misunderstanding to arise. Any arrangement between the builders and purchasers of such machinery is proper that promotes easy adjustment of commercial obligations; but when either party publicly discusses the results of their work the contract terms should be translated into generally accepted nomenclature.—Editor.]

[6] "The Surveyor and Municipal and County Engineer," Mar. 20, 1914.

More on Rock-Filled Dams

Sir—The recent failure of the lower Otay rock-fill dam has renewed an old subject of discussion which was very freely waged in the early nineties by the engineering profession, following the failure of the Walnut Grove dam in Arizona, Feb. 22, 1890.

In this early discussion the most generally accepted conclusion was that a properly designed rock-fill dam, constructed of suitable materials and properly safeguarded, was a safe and satisfactory type and should be adopted wherever it could be used with commercial economy.

On July 7, 1890, the writer as consulting engineer of the Escondido Irrigation District made a report advising as one of the necessary features of the proposed works the construction of a rock-fill dam at the point where it was later built. This proposed type of dam raised a local tempest, as the Arizona disaster was fresh in the minds of the district residents. To a great degree and largely through the presentation of an analysis of the resistance of the Walnut Grove dam when subjected to a heavy overflow for which it was not designed the writer successfully satisfied criticism and objections.

The original design of the Escondido rock-fill dam provided for a top elevation 110 ft. above bedrock and a top width of 10 ft., with a coefficient against sliding of 3.5; it provided also an overflow weir at an elevation of 102 ft. above bedrock, located in a saddle remote from the dam.

The plans called for hand- and derrick-placed rock in both faces—the upper face 20 ft. thick at the bottom and 10 ft. thick at the top, the lower 28 ft. thick at the bottom and 5 ft. at the top. The core was to be of loose rock free from earth, sand, gravel and spalls. The upper face was to be covered with concrete thoroughly anchored into the rubble.

The present dam was completed five years later with the following changes from the original plans: The height was reduced to 76 ft., the upper face was planked, the lower slope was flattened in its lower half, and all rock was dumped loosely in place, excepting for a 10-ft. facing of hand rubble on the upstream side. The overflow weir was placed at the north end of the dam and constructed as a canal 20 ft. wide and 5 ft. deep.

The writer was not in charge of construction, but has always considered the principal element of danger in the revised plans to be found in the restricted spillway or waste weir.

During the recent storms this dam was overtopped for some 6 hr. with an overflow sheet of water said to have had a depth of 2 in. at the abutments and a greater depth at the center of the structure, where for some 60 ft. the

depth was in excess of 12 in. The rock-fill settled at some points as much as 1 ft. and a small quantity of rock on the lower face was displaced. That this structure is now standing is due solely to the fact that the body of the dam was composed of large rock with maximum voids free from spalls, gravel, sand and earth.

It is reported that the owners of the Escondido reservoir will soon build the dam to the elevation of the original design and take advantage of the low saddle in the watershed line to construct a long and safe waste or overflow weir.

The writer is not surprised that the Lower Otay dam failed immediately the lake level overtopped the structure, and he further believes that a failure with the water level at the spillway elevation would at least have been logical.

No rock-fill dam having a cross-section within the limits of commercial investment would possess inertia sufficient to withstand for any great length of time an overflow volume of water which could not readily escape through the voids of the structure. Even with open voids the structure would quickly fail, by erosive action of the water, if the overflow volume exceeded the absorption capacity of the rock-fill.

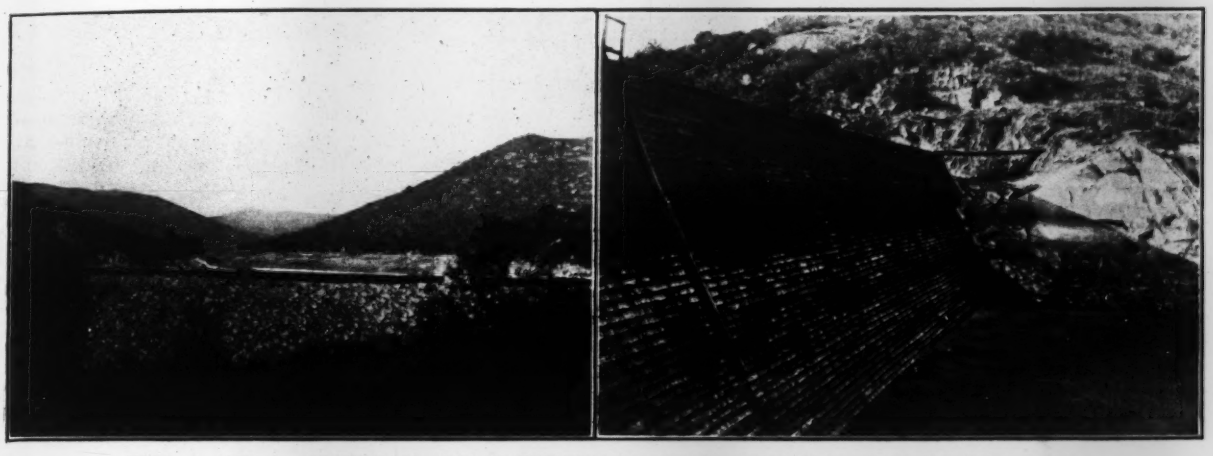
It is well to remember that disintegration of the rock mass in this type of dam is constantly at work, with the result that settlements and readjustments of, and within, the mass produce a tendency of the whole structure to slide downstream, when subjected to the increasing thrust produced by the rapidly rising reservoir waters.

From an examination of photographs representing the progress of work during the construction of the dam one gets a strong suggestion that too much fine material was allowed within the structure. This fine material coupled with the product of disintegration would largely reduce the voids in the mass and constitute a menace needing only a comparatively small quantity of water in the lower half of the dam to produce "blowouts" of such magnitude as to result in immediate destruction.

It is evident that the readjustments within this dam, as previously described, were sufficient to shear the thin composite "plate-metal and 2-ft. concrete" core wall and, further, that the leakage through the fractured core wall quickly saturated the rock mass below and produced the complete failure of the structure.

Eyewitnesses of the 6-hr. overflow of the Escondido dam state that at the time of maximum overflow all water was absorbed by the voids of the mass within a few feet of the top of the dam and only appeared again at the base of the structure, where it left the rock-fill without producing erosive action.

In further support of the principle that rock-fill dams never should be used as overflow weirs the writer desires to call the attention of engineers to the monumental failures that may be found in many of the mountain cañons of western America. These colossal dikes, or "hog's backs," of broken rocks which in past geological times have slid into the cañons from the adjacent mountain mass have completely dammed the cañons and created lakes or reservoirs, in some cases hundreds of feet in depth; but in every case, so far as the writer has observed, these "natural rock-fill dams" have been cut through by overflow water and have failed as dams, notwithstanding the fact that in cross-section they were many times more conservative than any artificial structure yet erected.



FIGS. 1 AND 2. UPSTREAM AND DOWNSTREAM VIEWS OF ESCONDIDO ROCK-FILL DAM

The lessons to be learned from the recent action of these two California rock-fill dams are of great value to engineers and may be briefly stated as follows:

Rock-fill dams should never be constructed unless supplemented by the liberal use of "waste weirs" to insure against the overtopping of the structure.

Only hard, durable, crystalline rock, in large pieces, free from spalls and other fine materials, should be allowed in these dams.

These structures should be designed with a coefficient against sliding of not less than 3.5 and with a core wall at or near the upstream face of the dam.

Los Angeles, Calif., Apr. 1, 1916. F. E. TRASK.

A Scenic Automobile Highway to the top of Mt. Whitney is the ambitious scheme of the Board of Supervisors of Kern County, California, which recently adopted resolutions asking the Board of Supervisors of Los Angeles County and the Automobile Club of Southern California to coöperate in asking the assistance of the Federal Government to that end. Mt. Whitney is the highest mountain peak in California, its top being at 14,501 ft. above sea level. The length of the proposed highway from Bakersfield is 150 mi. Kern County agrees to put the present road from Bakersfield to Weldon in good condition, provided the Federal Government will construct the remaining 75 mi. of road from the town of Weldon to the summit of Mt. Whitney. This portion of the road, which is estimated to cost $343,000, will lie within the Sequoia National Forest Reserve.

Coating Steel at Grand Central Terminal With Cement Gun

By W. F. Jordan*

SYNOPSIS—Over 1,000,000 sq.ft. of 2-in. concrete fireproofing applied to steelwork of the Grand Central Terminal at New York with cement gun. Resulting mortar hard and effective. Cost lower than by hand placing.

The tracks in the Grand Central Terminal at New York City are on two levels. The upper level is carried on a steel and concrete structure, the general design of which is shown in the photographs. The concrete jack arches were built before the fireproofing work was started, so the operating tracks on the upper level could be laid. This left the soffits of the stringers, the lower part of the girders, brackets and columns to be protected later by fireproofing.

A minimum thickness of 2 in. of fireproofing was required, and it was important that the fireproofing adhere closely to the steel to protect it from moisture and rusting as well as from fire. The cement gun was chosen to do the work, which was started in 1911 and is still continuing. Up to date about 1,000,000 sq.ft. of surfacing has been applied.

After the steelwork was cleaned and repainted wire lath was placed about 1 in. from the metal. This lath was made up of Clinton electrically welded cloth of No. 12 wire, 2x2-in. mesh and galvanized. The cloth was tied to ⅜-in. rods, which in turn were fastened to the structure. On the bottoms of girders the rods were bent over the flanges; on the sides of girders and brackets the rods were passed through holes in the stiffeners and tied between with No. 6 wire through holes punched in the web of the girders. The rods were spaced from 12 to 15 in. apart.

The cement guns are mounted on a low push-car so that the door by which they are charged comes just above the floor of a flat-car. This push-car is placed next to a standard flat-car on which is a shaking screen. The cement is kept on this car, and the cement and sand are mixed on it. The sand is brought in on cars and is placed immediately behind the mixing-car. The cement and sand are mixed, passed through the shaking screen and then shoveled into the hopper of the cement gun as required. The gun blows the dry mixture through a 1-in. hose, and the nozzleman directs the stream onto the steel to be protected. Water is taken to the nozzle through a separate hose and is sprayed into the mixture at the nozzle, the nozzleman having a valve to control the amount of water. He plays the stream back and forth until the desired thickness is obtained.

It is found best to apply the coating in two layers. Trains are running on the tracks laid on the roof of the structure, and the jar from the moving trains was apt to shake the fireproofing off, if it was all put on at one time; but when put on in two layers, there was no trouble. On the sides of the girders there is a tendency for the sand to fall down into the angles of the structure and make porous places next to the steel. This was prevented by shooting the top of the bottom flange first, filling the angles before the space between them.

After shooting, a mason trims off the irregularities and floats the surface. In applying the mixture a part of the sand rebounds and is lost; a part is shot by the edges of the structure, and some is scraped away by the mason trimming off the irregularities. It is estimated that from 15 to 20% is lost.

The coating has a minimum thickness of 2 in., but owing to irregularities in the steel and rounding out the fireproofing in the angles the average thickness is from 2½ to 3 in.; generally it is about 3 in.

The mixture used is 1 cement and 3 sand. This makes a very hard and dense coating. To test the density, some of the mixture was wetted and applied with a trowel as mortar; after setting, this was found to weigh 124 lb. per cu.ft. Some of the fireproofing applied with the cement gun was weighed after setting and was found to weigh 144 lb. per cu.ft. It is very hard and when set requires a chisel to cut it. No cracks have appeared in the fireproofing except in the pipe subway, where there were hot pipes at the time the work was put on; the heat dried out the mortar too quickly, producing hair cracks.

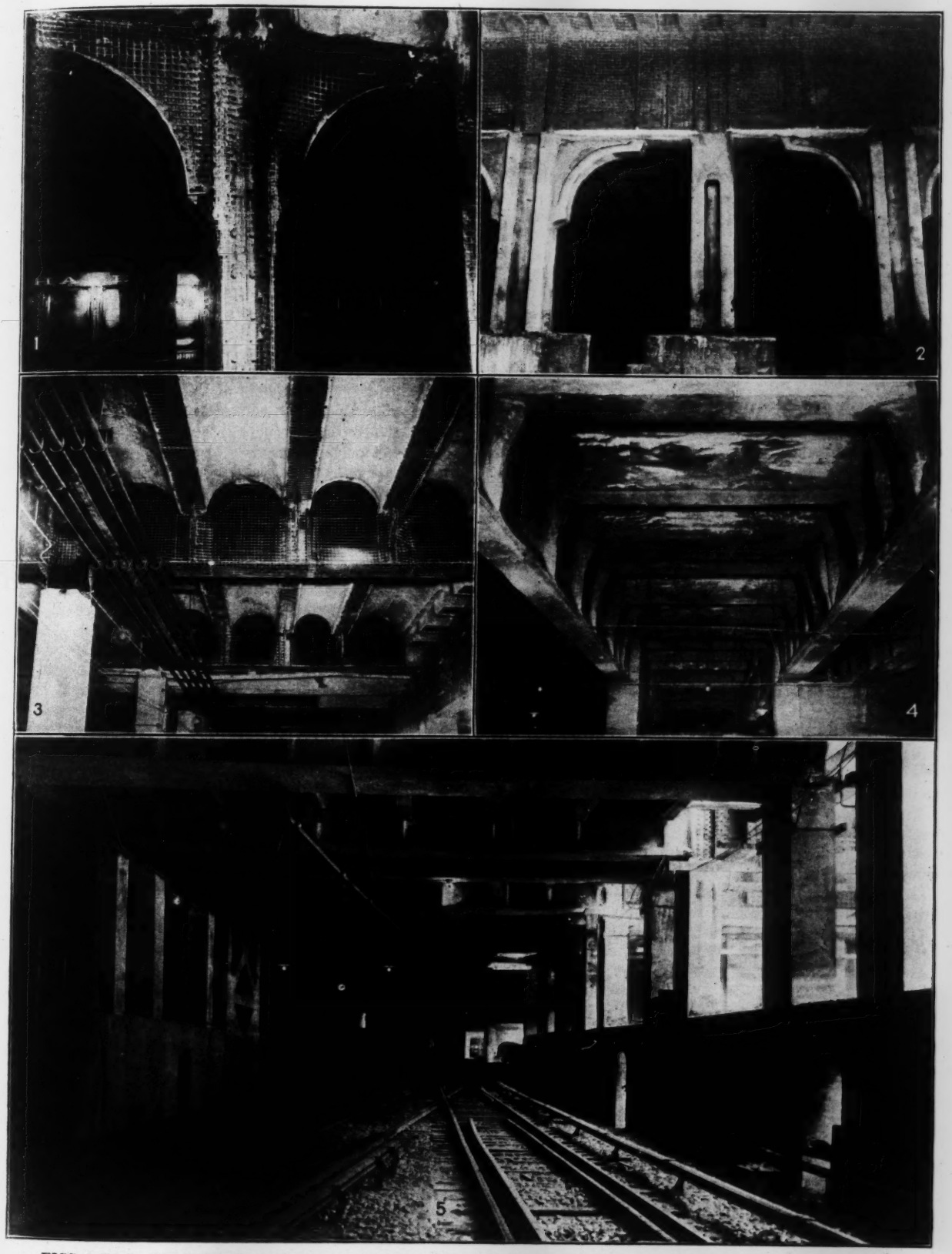
The machine has been used successfully with 250 ft. of hose, but an effort is always made to keep the hose as short as possible. During the early operation the best hose available wore so rapidly that with a 100-ft. length on a machine the cost of hose ran as high as $5 per day. Manufacturers now make a hose adapted to the service, and the cost has been reduced to 75c. per day. Care is taken to avoid short bends, and the hose is turned end for end occasionally.

The area covered per gun per day has averaged 600 sq.ft., and where conditions were very favorable as much as 1,200 sq.ft. has been done in a day. To cover 600 sq.ft. the material required on an average has been 94 bags of cement and 9.4 cu.yd. of sand. The most material that has been passed through a machine in one day is 125 bags of cement and 14 cu.yd. of sand. When the large areas around 1,200 ft. were obtained, it was in places where the steel had fewer irregularities and the average thickness was less than for the general run of the work.

The sand for the cement gun should be sharp, not too coarse, and rather dry; very dry sand and wet sand do not work well. Care is taken that pebbles do not get into the machine. They are likely to clog the hose.

The reinforcement is put on by contract. The concreting is handled by a foreman with a man to operate the machine, four laborers to mix and screen the ingredients and charge the machine, a nozzleman to apply the mixture and a mason to finish the work. Frequently some additional work is needed, and men are called from other places to attend to it. Much of the work has been over tracks in revenue operation, and an operating watch-

*Manager, Grand Central Terminal Improvements, New York Central R.R., New York City.



FIGS. 1 TO 5. FRAMEWORK OF GRAND CENTRAL TERMINAL FIREPROOFED WITH CEMENT-GUN MORTAR

Fig. 1—Wire mesh on column before mortar application. Fig. 2—Column coated with mortar, girders and floor with reinforcement in place. Fig. 3—Complicated jack arch construction partly coated. Fig. 4—Completed ceiling coating. Fig. 5—Completed track bay, showing how close to traveled tracks work had to be done

man is required to protect the workmen. Some of the sand drops into the ballasted track below, and there is a charge for the cost of cleaning the ballast. There is also some expense for protecting the third rail The machine uses 100 cu.ft. of free air per minute, compressed to 40 lb. Four men are employed to handle the scaffolding, and the cost of this part of the work is divided between the painting and the fireproofing. The accompanying table gives some average costs of the cement-gun work. An attempt was made at putting the fireproofing on by hand. The cost was 35c. per sq.ft. exclusive of the lathing.

COST OF FIREPROOFING FOR A DAY'S RUN OF 600 SQ.FT.

Scaffolding, charged to fireproofing		$7.20
Labor		26.00
Material:		
94 bags of cement at 32c.	$30.08	
9.4 cu.yd. of sand at 50c.	4.70	
		34.78
Compressed air		3.50
Watching, cleaning ballast and protecting third rail		3.25
Supplies and repairs		2.00
Switching service		1.00
Interest and depreciation, superintendence, office expense and incidentals		13.27
Total		$91.00

$91 ÷ 600 = 15¼c. per sq.ft.

In the accompanying views the various stages of the structure are shown from bare steel, with wire attached, to the finished fireproofed steelwork.

Some Important Details of Land Drainage Work

In a report on drainage in the lower peninsula of Michigan, A. D. Morehouse, Drainage Engineer of the United States Department of Agriculture, makes very pertinent comments on the various problems of construction and maintenance of land drainage systems in general, using the practice in that locality for the basis of study.

There is little uniformity in the cost of trenching and laying tile, especially in contract prices for county work. Figures show a range of 36c. per rod for 8-in. tile drain (average cut, 1½ ft.) to $1.69 for 8- and 12-in. tile (4-ft. cut), and $4.34 for 10-in. tile and 6-ft. average cut. The difference is far beyond what might be accounted for by variation in soil or the presence of timber.

The accompanying table gives an interesting review of the distribution of cost on 84 drains aggregating 131 mi. in one county. The total cost was $152,438, averaging $3.63 per rod, or $1,161.44 per mi. Of the total, 83.26% was for construction, 9.67% for office and miscellaneous expenses, and only 3.41% for engineering.

DISTRIBUTION OF COST OF TILE DRAINAGE

	Per Cent.		Per Cent.
Surveys and plans	3.13	Additional work	2.91
Printing notice of letting	1.90	Office work	2.24
Construction	47.15	Probate fees and exp. of commrs. of jury	2.54
Tile and hauling	29.80	Damages allowed	0.05
Culverts and arches	5.75	Balance on hand	4.22
Inspection	0.28		

From a study of ditch profiles it is evident that the desire to save excavation by keeping the grade as nearly as possible at a uniform depth below the surface has been a ruling factor in ditch design. In this way the hydraulic features are overlooked. The result is insufficient depth of outlet for lateral ditches and the running of a steep grade into a flatter one. This latter feature causes slacking of the velocity of the water and deposition of silt, and also the danger of having too little capacity in the ditch with the flatter gradient. The first consideration should be to secure the desired drainage results by a ditch that will be as nearly self-maintaining as possible. The amount of excavation necessary should be regarded as a secondary matter.

In few cases were the size and fall of the ditch so designed as to give it a capacity determined from the topographical and geological character of the watershed, in connection with the rainfall data. Such details have been simply matters of the drain commissioners' judgment or convenience, rather than of an engineer's study and calculations.

The efficiency of drainage works is not gaged by their ability to handle rainfalls of average amount, but they must be of sufficient capacity to take care of the maximum storms which (while occurring at infrequent intervals) must be provided for within a short space of time, if crop injury or destruction is to be prevented. It is probable that a runoff of ¼ to 1 in. of water from the watershed in a 24-hr. period would be the ordinary range for the varying topography and soils of this section. The exact amount must be determined after consideration of the local conditions.

There is a marked tendency to make ditches wide and shallow rather than deep and narrow. Except in cases of necessity, where sand, quicksand or rock would be encountered, the latter are more desirable. They furnish better and deeper drainage, are more easily maintained, have greater carrying capacity for a given amount of excavation and require less width of right-of-way. The berm at the top of the ditch should be of such width that when the banks of the ditch take their natural slopes none of the spoil will fall or be washed into the ditch. For ordinary dredged ditches of moderate depth this width will be from 8 to 10 ft. as a minimum.

While ditches should be as straight as practicable, they should usually follow natural drainage depressions or channels except where these latter may be straightened conveniently and economically and without danger of flooding low lands lower down the stream.

The cleaning of ditches is often left for a number of years (averaging 10 yr. in one lot of ditches investigated, with a maximum of 36 yr.). In many cases it is a virtual reconstruction of the ditch. Under ordinary circumstances, in the section under consideration, a new ditch properly constructed should require little attention or repair for three years, but it should afterward be given annual attention. In case of any exceptional obstruction such as the overthrowing of trees into the ditch by storms, the matter should be remedied at once without waiting for the annual overhauling.

It is probable that from 1 to 2% of the original cost of a ditch, spent annually in the removal of weeds, silt, logs, trees or other obstructions, will keep the ditch up to its original efficiency and prove of benefit to the adjacent lands.

Much damage to new ditches has been caused by animals, owing to the failure of landowners to fence the ditches at the time of construction. It has been suggested that it would be proper to make the original cost of such fences a part of the cost of the ditch. After the fences are built, the landowner should be required to maintain them.

In many cases county drains have been impaired or ruined by lateral tile drains or ditches entering the main ditch without any protection being provided for the latter.

News of the Engineering World

Nashville Engineers Active in Society Work

The Nashville, Tenn., section of the Engineering Association of the South has ended a one-week membership campaign, which added 55 new members, bringing the total to 140. The campaign was carried on by two teams of six men each, called the "Squares" and the "Triangles." The Squares, led by M. A. Lightman, attained the higher record, and was presented with a loving cup inscribed with the name of the successful campaigners.

The section has also offered a medal to the member who writes a paper for the local press which, in the opinion of judges, does the most good for the society. The subjects are to be submitted to the society for approval and should aim to show what engineers (and particularly the society) are doing for the public.

A movement recently started by the section has as its aim the inclusion of at least one engineer in the proposed convention to revise the state constitution. Other engineering societies of the state are to be asked to aid in having engineers nominated delegates to this convention. The section will appoint a committee to work with other engineering organizations with the idea of submitting suggestions to the constitutional convention.

New York May Use More Meters

Some of the most serious limitations on the use of water meters in New York City have been removed by a legislative act signed by Governor Whitman on May 19. The principal changes effected or made possible by the act have been summarized for *Engineering News* by Delos F. Wilcox, Deputy Commissioner of Water-Supply of New York City, as follows:

1. The Department, with the approval of the Board of Aldermen, may assume the expense of furnishing, installing and maintaining meters, thus changing the system from private ownership to city ownership of meters.
2. The Department, with the approval of the Board of Aldermen, is authorized to acquire meters now owned by the property owners and to assume the expense of keeping them in repair.
3. The right to demand the installation of a meter is extended to every property owner connected with the public water supply.
4. The city is authorized to establish annual service charges and minimum charges in connection with a schedule of meter rates.
5. The Department is authorized to apply the revenue from service charges to the financing of the meter system.
6. The method of estimating the amount of water used while the meter is out of repair is made more elastic.
7. The consent of the Board of Aldermen to the installation of meters on residential premises will no longer have to be given for all or none, but may be given for "any or all," thus enabling the Board of Aldermen, if it sees fit, to classify residential premises and give its consent to the installation of meters in one or more classes.

A review of the New York meter situation, based on an official statement, appeared in *Engineering News* for April 6, 1916, page 657, accompanied by editorial comment on page 665.

Engineers Hold Nationwide Telephonic Session

A memorable national meeting was held by the American Institute of Electrical Engineers on May 15. The meeting involved simultaneous sessions in San Francisco, Chicago, Atlanta, Philadelphia, Boston, Salt Lake City, Denver and New York, and they all were tied together by means of long-distance telephone circuits, 5,000 receivers being employed by the large and scattered attendance. The hour of meeting varied in the different time zones to correspond with 8:30 p.m. Eastern time. The meeting commemorates the achievements of the Institute's membership in the field of communication, transportation, lighting and power.

At exactly 8:30, President John J. Carty in New York called the great meeting to order and requested reports from the section officers presiding in the other cities. Each responded with a statement that his city was in session with such and such an attendance. The attendance was as follows: Atlanta, 500; San Francisco, 750; Chicago, 1,000; Boston, 900; Philadelphia, 850; New York, 1,100. Denver and Salt Lake City, though able to hear the proceedings of the meeting, were not able to take part otherwise. During the course of the evening these two cities telegraphed their attendance, which was 40 for Salt Lake City and 50 for Denver, making a total of 5,200 throughout the country.

At 9 o'clock, Eastern time, the telephonic connections were discontinued for a half-hour to permit local addresses to be delivered. When the national character of the meeting was resumed, general greetings were exchanged and national airs rendered by phonograph. Following this, Dr. Michael I. Pupin addressed "The Engineering Profession." Following this address was a general greeting from Honorary Secretary Ralph W. Pope.

Aberdeen Buys 2,000,000 Ft. of Staves for Water Conduit

Wood staves for the new water-supply conduit of Aberdeen, Wash., will be supplied by local mills through the Pacific Lumber Agency at $25 per 1,000 ft. An earlier report (*Engineering News*, May 4, 1916, p. 869) stated that 1,000,000 of the 2,000,000 ft. in this order would be furnished by the Vance Lumber Co. for $23 per 1,000 ft., f.o.b. Malone, Wash., but that company has released the city from obligation to it. Freight at $1.20 per 1,000 ft. and the greater cost of inspection at Malone, it is held, will just make up for the increased contract price. Under the contract just awarded the staves may contain one knot 1 in. or less in diameter every 4 lin.ft., and these knots may be on the edge of the staves. For $3 more per 1,000 ft., knots only ½ in. in diameter, extending only part way through the stave, with none at the edge, might have been obtained. L. C. Kelsey,

City Engineer, advised the cheaper staves, stating that they would make a durable pipe line and that the same grade had been used for the Cedar River conduits, Seattle. Two councilmen voted against the cheaper staves. The conduit will be 28 in. in diameter and 22 mi. long. Bids for laying it are to be received on May 31.

Union Station at Denver, Colo., Nearing Completion

The reconstruction of the union station at Denver, Colo., by the Denver Union Terminal Railway Co., the plans for which were described in detail in *Engineering News*, Apr. 1, 1915, is now nearing completion. The accompanying views were taken at the end of April, 1916. The work is complicated by the fact that the station building, yard and tracks have to be kept in operation during the changes. The new station building has been completed for some time, and work on the rearrangement and elevation of tracks and the construction of subway passages to the train platforms is progressing rapidly.

Fig. 1 shows the station building. The central portion (165x140 ft.) is new. It contains a large waiting room, the full height of the structure, with galleries around the outside leading to the offices. The older portion of the structure, forming the wings, is used for baggage and express, dining room, etc.

The reconstruction of tracks and yard (a plan of which was shown in *Engineering News*, Apr. 1, 1915) involves the handling of 55,000 cu.yd. of material, the tracks being raised about 3 ft. There are about 150 passenger train movements a day to be taken care of during these

FIGS. 1 TO 4. VIEWS OF THE NEW UNION STATION UNDER CONSTRUCTION AT DENVER, COLO.

Fig. 1—Finished front; old portion at right. Fig. 2—Concrete mixers in yard. Fig. 3—Construction work in yard; station at right. Fig. 4—Top of umbrella sheds.

changes. While essentially a terminal station the tracks are continuous through the station yards. Figs. 2 to 4 show some of this reconstruction work.

The umbrella-type of platform shelters of reinforced concrete form an interesting feature of the station. They are very similar to those recently erected by the Southern Pacific Ry. at Los Angeles and at the 3rd and Townsend St. station, San Francisco. The Denver station contains 6,700 lin.ft. of these shelters. Expansion joints are provided approximately 340 ft. apart, a sheet of galvanized metal being placed over each opening to keep water from dripping to the platform below. Fig. 4 shows some of these sheds completed.

The sheds are built of separately cast units placed by means of a locomotive crane. The pedestals were cast in place. On these were set the precast posts, and on the posts the roof slabs, precast in 20-ft. units. The posts are joined to the pedestals by 1 in. of cement grout.

The other views show the construction of the subways, of which there are three—one for passengers and one each for baggage and for mail and express matter. These are all to be served by electric elevators. The forms for the concrete are of wood, made in large units, and are moved from place to place by locomotive cranes. For concreting these subways small mixers erected on timber towers are used, and the concrete chuted into place (see Figs. 2 and 3).

The Denver Union Terminal Railway Co. is composed of six railways, represented by the following: J. G. Gwyn, Chief Engineer of the Denver & Rio Grande R.R. (chairman); E. F. Vincent, Chief Engineer of the Colorado & Southern Ry.; C. F. W. Felt, Chief Engineer of the Atchison, Topeka & Santa Fe Ry.; T. E. Calvert, Chief Engineer of the Chicago, Burlington & Quincy R.R.; C. A. Morse, Chief Engineer of the Chicago, Rock Island & Pacific Ry., and R. L. Huntley, Chief Engineer of the Union Pacific Ry. Until his death recently F. C. Cowan, Chief Engineer of the Colorado & Southern, was chairman of this engineering board. C. C. Post, former Division Engineer of the Union Pacific Ry., is Construction Engineer. Stocker & Fraser, of Denver, are the general contractors; the Van Sant-Houghton Co., San Francisco, has the contract for the concrete umbrella sheds; and Hinchman & Renton, Denver, are contractors for the piping and drainage of the yard. The work was begun in September, 1914. The total cost of the improvements approximates $4,000,000.

A Blast Demolished a Restaurant in Akron, Ohio, on May 15, killing 9 persons and injuring 28 others. The cause of the accident was blasting in an excavation for a new building next door to the Crystal restaurant on Main and Quarry St. The building collapsed immediately after the blast. There were 35 diners and 15 employees in the wrecked building. The property damage is estimated at $25,000. The building is of one story, having a two-story false front, and was built in 1886. It was only 20 ft. wide. The false front toppled backward, falling upon the collapsing roof. Three weeks ago another blast damaged a part of the wall of the Strand Theater, north of the excavation. In the report of three state inspectors—made public on May 19—the excavating contractor is blamed for the accident. The careless blasting took place in spite of safety orders issued by the city.

Road Institutes in 30 Alabama Counties, held recently under the State Highway Commission (W. S. Kellar, Engineer), were attended by 1,745 people, of whom one-third are actually engaged in roadwork.

The Building-Trade Strike in St. Louis was settled on May 18 by a compromise which gave the hodcarriers and building laborers an average of about 2½c. an hour increase, and a further agreement that there would be no discrimination on account of union affiliation.

Philadelphia Public Improvements to the amount of $115,000,000 were authorized by a vote of 3 to 1 on May 16, when the electorate approved the issue of bonds submitted by the Councils. This amount is to be distributed as follows: Transit development, $57,100,000; new diagonal parkway and boulevard, $9,000,000; port improvements, $5,000,000; grade-crossing removal in South Philadelphia, $5,640,000; sewage treatment, $3,200,000. General improvements take up the remainder.

The Humphreys Bill providing appropriations for the flood-control of the Mississippi and Sacramento Rivers and establishing precedent for Federal expenditure in all flood-control work passed the House of Representatives on May 17. The bill is the same as noted in "Engineering News," May 18, 1916, p. 954, except that an amendment was carried providing that local interests along the Mississippi should contribute one-half as much as the Federal government instead of the one-third specified in the original bill.

Municipal Improvements at Erie, Penn., to cost $1,445,000 were indorsed by a 4 to 1 popular vote on May 16, when bonds for that amount were authorized. Most of the sum will go for stream control, $950,000 being devoted to the improvement of Mill Creek and $200,000 to Garrison Run, a small stream in the eastern part of the city. Grade crossing elimination will take $225,000 of the total and storm sewers $70,000. B. E. Briggs, city engineer, writes "Engineering News" that work on plans for the improvements will be started at once in order that contracts may be let at the earliest possible date.

Limits of New York Harbor—Suggestions respecting the lines designating the waters of New York Harbor, as contemplated by the Seaman's Act, have been made by the committee appointed for the purpose and the report has been approved by Secretary of Commerce Wm. C. Redfield. The committee consisted of Assistant Secretary of Commerce E. F. Sweet, chairman; A. L. Thurman, solicitor of Department of Commerce, and Supervising Inspector Gen. George Uhler, of the Steamboat Inspection Service. Suggestions and opinions of interested persons were received by this committee at a hearing in New York City on Apr. 26, a unanimous report of which has now been made. The harbor limits suggested are as follows:

The limits of the harbor seaward shall be an imaginary line drawn from Navesink Lighthouse, 25½° true (NE. ⅞ N. mag.), to the Life Saving Station on Rockaway Beach, and all waters inside of this line to the northward and westward shall be considered as within the limits of New York Harbor.

The limits of the harbor eastward and meeting Long Island Sound shall be an imaginary line drawn from Eatons Point Lighthouse on the north shore of Long Island, N. Y., 9½° true (N. by E. ¾ E. mag.), through the lighthouse on Pecks Ledge to the Connecticut shore east of the Saugatuck River, and all the waters inside of this line to the westward shall be considered as within the limits of New York Harbor.

Another Scholarship in Chemical Engineering—The Chemists' Club of New York has announced the establishment of another scholarship fund, the income from which, approximately $400 per year, is to be devoted to assisting financially deserving young men to obtain education in the field of industrial chemistry or chemical engineering. This scholarship has been endowed by William F. Hoffmann. Its benefits will be open to properly qualified applicants without restriction as to residence and may be effective at any institution in the United States which may be designated or approved by the Chemists' Club. Applicants must, as a minimum qualification, have completed a satisfactory high-school training involving substantial work in elementary chemistry, physics and mathematics and present a certificate showing that they have passed the entrance-examination requirements of the College Entrance Examination Board or its equivalent. Preference will be given to young men who have supplemented these minimum qualifications with additional academic work, especially in subjects which will form a suitable foundation for the more advanced study of applied chemistry and chemical engineering. All inquiries should be addressed to the Hoffmann Scholarship Committee of the Chemists' Club, 50 East 41st St., New York City. Applications for the academic year 1916-17 should be in the hands of the committee on or before June 1, 1916. William F. Hoffmann, the founder of the Hoffmann scholarship, is president of the American Oil and Supply Co., Newark, N. J. He is one of the founders of the Chemists' Club of New York.

PERSONALS

Mr. O. J. S. Ellingston, Assoc. M. Am. Soc. C. E., formerly City Engineer of Sherman, Tex., has been appointed City Manager of that city.

Mr. L. W. Cady has been promoted to the position of Efficiency Engineer of the Division of Light and Heat of the City of Cleveland, Ohio.

Mr. A. L. Mohler, President of the Union Pacific R.R., has announced his retirement from active service, to take effect on July 1. Although he retires as President, he will be identified with the system in an advisory capacity.

Mr. William von Phul, M. Am. Soc. C. E., Vice-President and General Manager of the American Cities Co., and a member of the firm of Ford, Bacon & Davis, will succeed Charles N. Black as Vice-President of the United Railroads of San Francisco.

Messrs. Clarence P. Day and S. H. Phelan announce that the firm of Day & Phelan, Civil and Landscape Engineers, with offices in the First National Bank Building, Santa Barbara, Calif., has been dissolved. The business of the firm will hereafter be conducted by S. H. Phelan, at the same address.

Mr. H. H. Maxfield, M. Am. Soc. M. E., recently promoted to Superintendent of Motive Power of the Western Pennsylvania division of the Pennsylvania R.R., was born in 1873. He was educated at Stevens Institute of Technology and entered the service of the Pennsylvania R.R., as apprentice in the Meadow Shops, in 1865.

Mr. A. B. Gates, formerly Assistant Electrical Engineer of the Sanitary District of Chicago, and recently an assistant to Gardner S. Williams, consulting engineer, in valuation and accounting work, has entered the engineering department of the Commonwealth Edison Co., Chicago, as Engineer on Generating Stations.

Mr. George C. Andrews, Assoc. M. Am. Soc. C. E., until recently Resident Engineer of the New York State Barge Canal at Lockport, N. Y., has been appointed Supervising Engineer, Bureau of Water of the City of Buffalo, under the commission form of government. Mr. Andrews is a graduate of Lafayette College.

Mr. John Hugher, for 13 years General Manager of the Steel Products Co., a subsidiary of the United States Steel Corporation, has been made Assistant to President Farrell, succeeding W. D. Terley, who has been made Vice-President and General Manager of the new Canadian steel works to be built by the corporation.

Mr. Francis H. Wisewell, Jr., formerly connected with Westinghouse Church Kerr & Co., New York City, has been appointed Chief Engineer of the Gifford-Wood Co., Hudson, N. Y. Previous to his connection with Westinghouse, he was Mechanical Engineer with the Continental Car and Equipment Co., New York City.

Mr. F. L. Blendinger, Assistant to the Vice-President of the Lehigh Valley R.R., at New York City, has been made Vice-President in Charge of Operation and Maintenance of that railroad, and will continue to have his headquarters in New York City. Mr. Blendinger started railroad work, in 1878, as a messenger on the Erie R.R. at Little Valley, N. Y.

Mr. Frank R. Fisher, formerly in charge of the field work in the construction of the Market St. subway, Philadelphia, has been appointed engineer in charge of subway construction under the Philadelphia City Hall. His salary will be $6,000 a year. Mr. Fisher is a graduate of Lehigh University, and has been engaged in engineering work in Philadelphia for many years.

Mr. J. C. Allison, Assoc. M. Am. Soc. C E., formerly Assistant General Manager and Chief Engineer for W. H. Holabird, Receiver of the California Development Co. at Calexico, Calif., announces that on June 1 he will open offices for general engineering practice in the Anderson Building, now under construction, in Calexico. He will specialize in irrigation, drainage, silt investigation, municipal engineering and valuations.

Mr. Emil Diebitsch, M. Am. Soc. C. E., who was recently elected a Commissioner of the Town of Nutley, N. J., under the commission form of government, has been designated as Mayor-Commissioner of Public Affairs, Streets and Public Improvements, by his two fellow commissioners. Mr. Diebitsch is a graduate of Lehigh University. He is ex-President of the General Contractors' Association. In private business he is an engineer and contractor with offices in New York City.

Col. John Biddle, M. Am. Soc. C. E., Corps of Engineers, U. S. A., has been appointed Superintendent of the United States Military Academy, West Point, N. Y. He will relieve Col. Clarence P. Townsley, whose term has expired and who will be given a leave of absence before being assigned to duty in the Coast Artillery Division. Col. Biddle is a graduate of West Point in the class of 1881. He is at present stationed in Baltimore, Md., in charge of river and harbor improvement in that vicinity.

Mr. Francis L. Gilman, Fel. Am. Inst. E. E., Assistant General Superintendent of the Western Electric Co., Chicago, has been made Assistant Chief Engineer. Mr. Gilman graduated from Harvard University in 1895 and entered the employ of the New England Telephone Co. at Boston. In 1901 he took up switchboard engineering in New York City, and in 1906 was placed in charge of manufacturing for the company at New York City. In 1911 he was appointed General Manager of the Kansas City (Mo.) Telephone Co., and later became General Manager of the Bell Co. of Missouri.

Mr. Charles N. Black, Assoc. Am. Inst. E. E., has resigned as Vice-President and General Manager of the United Railroads of San Francisco. He will hereafter devote all of his time to the interests of Ford, Bacon & Davis, New York City, of which firm he is a member. Mr. Black has been located in San Francisco since 1907, previous to which time he was Vice-President and General Manager of the Metropolitan Street Ry., and Vice-President of the Kansas City Railway and Light Co. and the Kansas City Electric Light Co., all of Kansas City, Mo. He was Vice-President of the American Electric Railway Association from 1908 to 1913 and President of the Association in 1913 and 1914.

Mr. Fred C. Dunlap, M. Am. Soc. C. E., Consulting Engineer, former Chief of the Bureaus of Water and Filtration of the City of Philadelphia, has returned to the service of the city as Consulting Mechanical Engineer for advice in connection, particularly, with the design of certain parts of the Frankford Creek Intercepting Sewer. Mr. Dunlap entered the employ of the City of Philadelphia in 1898 as Assistant Engineer in the Bureau of Water, and was appointed Assistant Engineer in the Bureau of Filtration on designs in 1900. From 1902 to 1905 he was Engineer in charge of construction of the Torresdale filters. During 1906 and until May, 1907, he was Engineer of Design for the Jerome Park Filters in the Department of Water, Gas and Electricity, New York City, at which time he was appointed Chief Engineer of the Bureau of Filtration of Philadelphia. In September of the same year, when the Bureaus of Water and Filtration were combined, he was made Chief of both Bureaus and retained this position until his resignation in 1912.

Mr. Gamble Latrobe, recently appointed General Superintendent of the Philadelphia, Baltimore & Washington R.R. (Pennsylvania System) was born in Baltimore, Md., in 1866. He entered the service of the Pennsylvania in 1887 as levelman on construction work. The next year he resigned and entered the service of the Philadelphia & Reading Ry., but returned to the Pennsylvania in 1889, and was appointed Assistant Supervisor in 1890. In 1895 he was made Division Engineer of the Baltimore division, and in 1908 was appointed Acting General Agent and Superintendent, at Baltimore. Later in the same year he was made General Agent of the Philadelphia, Baltimore & Washington and the Northern Central Ry., and Superintendent of the Baltimore division of the Northern Central, which positions he has occupied until his present appointment.

Mr. Charles S. Krick, Acting General Superintendent of the New Jersey division of the Pennsylvania R.R., has been made General Superintendent of that division. He was born in Reading, Penn., in 1866, and is a graduate of Lafayette College (Class of 1887). After graduation Mr. Krick was employed as rodman in the assistant engineer's office of the Schuylkill division of the Pennsylvania. In 1903 he was appointed Assistant Engineer of the Eastern and Susquehanna divisions and was transferred to a similar position on the Philadelphia Terminal division during that same year. Three years later he was made Principal Assistant Engineer of the Philadelphia, Baltimore & Washington R.R. (Pennsylvania System). In 1912 he was promoted to Superintendent of the New York Terminal division, and when the Hudson and New York Terminal divisions were combined to form the Manhattan division, he was appointed Superintendent of that division. He was transferred to the Philadelphia Terminal division, in the same capacity, in 1914, and appointed Acting General Superintendent of the New Jersey division on Nov. 6, 1915.

Mr. Elisha Lee, General Superintendent of the Philadelphia, Baltimore & Washington R.R. (Pennsylvania System) has been promoted to the newly created office of Assistant General Manager of the Pennsylvania Lines East of Pittsburgh. He was born in Chicago, in 1870, and graduated from the Massachusetts Institute of Technology in 1892. The same year he entered the service of the Pennsylvania R.R. as rodman in the office of the Division Engineer of the Tyrone division. In 1903 Mr. Lee was appointed Assistant Engineer in the Maintenance-of-Way Department, and was advanced in 1907 to the position of Principal Assistant Engineer on the Philadelphia, Baltimore & Washington. In 1909 he was made Superintendent of the New York, Philadelphia & Norfolk R.R. (Pennsylvania System) and in 1911 appointed Assistant to the General Manager of the Pennsylvania Lines East of Pittsburgh and Erie. He was made General Superintendent of the Philadelphia, Baltimore & Washington in 1914. From 1912 to 1914 Mr. Lee served in the capacity of Chairman of the Conference Committee of Managers of the Eastern Railroads of the United States.

Mr. E. B. Ellicott, Fel. Am. Inst. E. E., has resigned as Chief Electrical Engineer of the Sanitary District of Chicago. He expects to devote considerable of his time to his ranch in Idaho and engage in consulting engineering in Salt Lake City. In 1885 Mr. Ellicott became connected with the Concordia (Kan.) Electric Light Co. In 1887 he was engineer and electrician for the Salina (Kan.) Gas and Electric Co., and the next year returned to Concordia as Superintendent of that plant. He entered the Western Electric Co., in 1890, and the following year was transferred by that company to Urbana, Ohio, in charge of the Urbana Electric Light Co. He was made Assistant Superintendent of Construction of the Western Electric Co. in 1892, which position he held until 1897, when he was appointed City Electrician of Chicago. He was granted a leave of absence in 1913, and as Chief Mechanical and Electrical Engineer of the Louisiana Purchase Exposition at St. Louis, designed and superintended the installation of a large part of the steam and electrical equipment of the Exposition. In 1905 he resigned as City Electrician of Chicago and was appointed Electrical Engineer of the Sanitary District.

OBITUARY

Willard T. Hough, Civil Engineer, Brooklyn, N. Y., died at Patchogue, N. Y., on May 8. He was 37 years of age.

Joseph J. Duffy, contractor, of Chicago, died on May 3, at the age of 56. He had handled a number of large contracts for the public works of Chicago.

James M. Mack, of the Mack Paving Co., Philadelphia, Penn., died suddenly at his home in that city on May 18. He had been in the contracting business since 1900.

Charles L. Negley, Engineer in the United States Forest Service, committed suicide at Portland, Ore., on Apr. 28. From a note which he left it is thought that he was despondent from ill health. Mr. Negley was a graduate of Cornell University and had been living in Portland about two years. He was 25 years old.

R. Mervin Lewis, Civil Engineer, killed himself during a fit of despondency at the home of a friend in Media, Penn., on May 10. He was Civil Engineer on the Media division of the Pittsburgh, Baltimore & Washington R.R. for five years until it was made part of the Maryland division recently. He was one of the officials sent to Wilmington, Del. His home was in Carlisle, Penn.

Frederick H. Meyers, Civil Engineer on the New York City Board of Water Supply, died at his home in Richmond Hill, N. Y., on May 21, at 45 years of age. Mr. Meyers was born in Allegheny County, Pennsylvania, and had lived in New York City and vicinity for several years. He was for a time engaged in the construction work on the northern aqueduct division of the Catskill Aqueduct.

Robert H. Ashworth, Civil Engineer, died at his home in Miami, Fla., on May 1. Mr. Ashworth had lived in Miami for about three years, previous to which he had been in the West engaged, with his son Frank K. Ashworth, on a number of projects, including the Colorado Springs water-supply and extensive irrigation works in Colorado and other Western states. The firm of Ashworth & Ashworth was first formed in Miami, but upon the declining health of Mr. Ashworth, his son made other business connections.

ENGINEERING SOCIETIES

AMERICAN IRON AND STEEL INSTITUTE.
May 26-27. General meeting in New York City. Secy., J. T. McCleary, 61 Broadway, New York City.

NATIONAL CONFERENCE ON CITY PLANNING.
June 5-7. Conference in Cleveland. Secy., Flavel Shurtleff, 19 Congress St., Boston, Mass.

AMERICAN WATER-WORKS ASSOCIATION.
June 5-9. Convention in New York City. Secy., J. M. Diven, 47 State St., Troy, N. Y.

SOCIETY OF AUTOMOBILE ENGINEERS.
June 12-16. Summer meeting (cruise on Lake Huron and Georgian Bay). Secy., Coker F. Clarkson, 29 W. 39th St., New York City.

MASTER CAR BUILDERS' ASSOCIATION.
June 14-16. Annual meeting, Atlantic City, N. J. Secy., Jos. W. Taylor, Karpen Bldg., Chicago.

AMERICAN INSTITUTE OF CHEMICAL ENGINEERS.
June 14-17. Eighth semiannual meeting. Cleveland. Secy., I. C. Olsen, Cooper Union, New York City.

RAILWAY SUPPLY MANUFACTURERS' ASSOCIATION.
June 14-21. Convention in Atlantic City. Secy., J. D. Conway, 2136 Oliver Bldg., Pittsburgh.

SOCIETY OF RAILWAY CLUB SECRETARIES.
June 16-17. Annual meeting in Atlantic City. Secy., H. D. Vought, 95 Liberty St., New York City.

AMERICAN RAILWAY MASTER MECHANICS ASSOCIATION.
June 19-21. Annual meeting, Atlantic City, N. J. Secy., Jos. W. Taylor, Karpen Bldg., Chicago.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION
June 19-22. Annual meeting in Charlottesville, Va. Secy., F. L. Bishop, University of Pittsburgh, Pittsburgh.

TRI-STATE WATER AND LIGHT ASSOCIATION OF THE CAROLINAS AND GEORGIA.
June 21-23. Annual convention at Isle of Pines, S. C. Secy., W. F. Stieglitz, Columbia, S. C.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
June 22-23. Meeting at Cleveland. Secy., Frank E. Sanborn, Ohio State University, Columbus, Ohio.

NATIONAL GAS-ENGINE ASSOCIATION.
June 27-29. Annual convention in Chicago. Secy., H. R. Brate, Lakemont, N. Y.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 27-30. Annual meeting in Pittsburgh. Secy., Charles Warren Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 27-30. Annual meeting in Atlantic City. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Penn.

The Washington State Road-Builders Institute held its first annual meeting last week on the campus of the University of Washington.

The American Electric Railway Association has announced that its 1916 annual convention will be held in Atlantic City, N. J., Oct. 9 to 13.

The Eighth National Conference on City Planning will be held at Cleveland, June 5 to 7. There will be six conference sessions at which the several phases of city planning will be discussed. There will also be a business session at 4:30 p.m., June 7, and a banquet on the evening of that day.

The National Association of Manufacturers held its 21st annual convention at the Waldorf-Astoria, New York City, May 15 to 17. Monday was devoted to the regular business of the organization, while Tuesday evening was given over to a foreign-trade round-table discussion. The legislative situation as affecting industries was discussed on Wednesday and the banquet was held in the evening, John Hays Hammond being one of the speakers.

The American Road-Builders' Association has scheduled its 14th annual convention, to be known as the Seventh American Good Roads Congress and the Eighth National Good Roads Show, for the week beginning Feb. 5, 1917, in Mechanics' Hall, Boston, Mass. This decision was reached by the board of directors at a recent meeting in New York City. The last time the association held its convention in New England was in 1904 at Hartford, Conn. The headquarters of the association are at 150 Nassau St., New York City.

The Brooklyn Engineers' Club will hold its second annual moonlight excursion on the Steamer "Mandalay" on June 15. The final inspection trip for the season will take place earlier, on Saturday, June 3, when the club in conjunction with the Municipal Engineers of the City of New York will visit the fortifications and Proving Ground at Sandy Hook, N. J. A special train will leave from the foot of Liberty St., New York City (C. R.R. of N. J.) at 12:30 p.m. Those desiring to take advantage of this opportunity should notify the secretary, Joseph Strachan, 117 Remsen St., Brooklyn, N. Y.

Appliances and Materials

Granite-Finish Terra Cotta

A new terra cotta building tile with a texture and general appearance of granite is being made by the Federal Terra Cotta Co., of New York City. The lower four stories of the new Printing Crafts Building, at 34th St. and Eighth Ave., New York City, are finished in this material.

* * *

Small Motor-Driven Humidifier

For service in rooms where a moist atmosphere is desired a small motor-driven mechanical humidifier is being made by the Normalair Co., Winston-Salem, N. C. Each unit comprises a disk and fan on the motor shaft; water flows from a regulating chamber to the center of the disk and is thrown against a copper grid around the circumference. The water is finely divided by impact, and the fan blows out the floating particles. The heavier particles drain down into a drip pan. The disk is 16 in. in diameter, and one unit will humidify, for instance, about 20,000 cu.ft. of textile-mill room.

* * *

Lusterless Wood-Finishing Coat

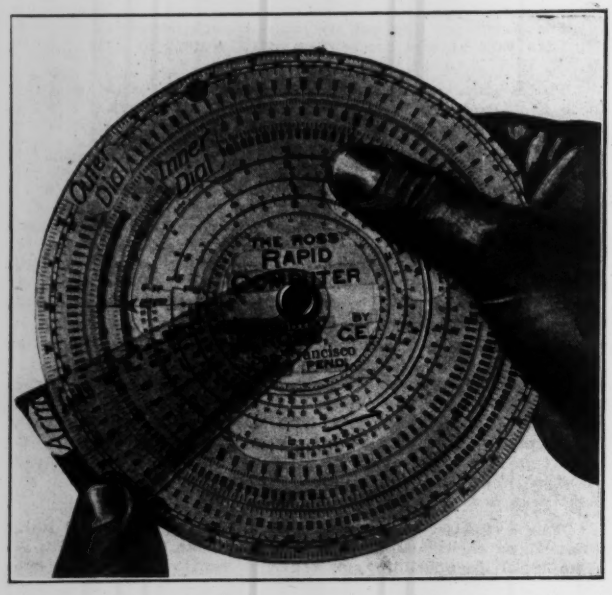
Under the trade name "Wonder Lac" a lusterless finish for stained wood is being made by the Bridgeport Wood Finishing Co., of New Milford, Conn. The new material has been compounded so as to be colorless and not to give any appearance of body—in response to a demand for a dampproof protecting coat which would not change the tone of stains or emphasize the finish.

* * *

New Circular Computer

A new circular-scale computing device has been devised by Louis Ross, of San Francisco, and is being made by the Computer Manufacturing Co., 25 California St., in that city. It has been aimed to secure a simple device handling a wider range of problems with fewer movements than possible with the slide rule. The computer is shown in the accompanying illustration. It is made in two sizes: 6-in. diameter, selling for $4.50, and 8-in. diameter, selling at $7.50. The instrument is made of weather- and wear-proof celluloid; it is flexible and has no glass parts; unequal shrinkage of parts, it is claimed, does not affect its accuracy. The two sizes correspond to the lower scales of a straight slide-rule, 16 and 21 in.

The scales are arranged to solve expressions like $a \times b \times c$, or $a \div b \times c$, and ordinary trigonometric calculations, with



ROSS "RAPID COMPUTER"

one setting of the dial. Beginning with the outermost, the scales on the outside dial are: (1) Degrees, 0° to 360°, for measuring angles; (2) Radians, 0 to 2 (6.283), for circular measure; (3) A scale of equal parts, 0 to 1,000, to read logarithms; (4) Scale N, for setting the first number and reading the answer. The scales on the inner dial are: (5) Scale M, like scale N, but numbered reversely, to set multiplier; (6) Scale D, like scale N, to set divisor; (7) Sines and cosecants, 6° to 90°; consines and secants, 0° to 84°; (8) Sines and cosecants, tangents and cotangents, 37′ to 6°; Cosines and secants, tangents and cotangents, 84° to 89° 23′; (9) Tangents and cotangents, 6° to 84°; (10) A scale of squares; (11) A scale of cubes. The conventional method of graduations has been replaced by a system of three and four digit numbers to facilitate direct reading. Each turn of either dial or reading arm performs an effective computing step—there are no blank movements. Full directions and detailed examples are furnished with each instrument.

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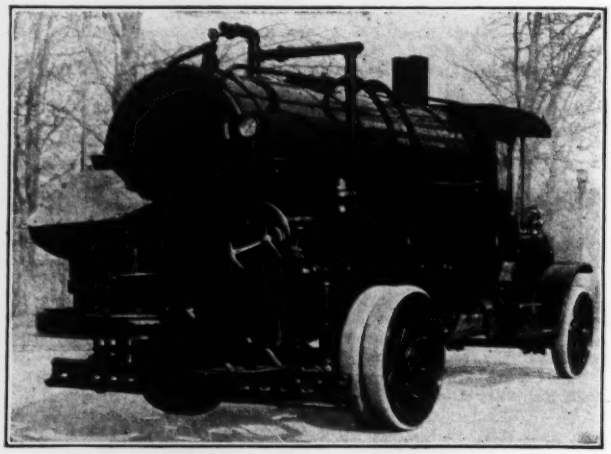
Road-Maintenance Truck

The combination road-maintenance and construction motor truck shown in the accompanying view has just been made by the White Co., of Cleveland, for the Department of Streets and Sewers of the City of St. Louis. Similar vehicles are



THE WHITE ROAD-MAINTENANCE TRUCK

being made for the Cities of San Francisco, Indianapolis, Fond du Lac, Wis., Lexington, Ky., for Onondaga County, N. Y., the Roads Preserving Co., of Cincinnati, and the Atlanta, Ga., Gas Light Co. The truck is the development of the same company's "good roads" truck; it is made in two sizes—one has an 800-gal. tank on a 3-ton chassis, and the other a 1,000-gal. tank on a 5-ton chassis. It is aimed to eliminate need of separate trucks for flushing and sprinkling streets, for applying light and heavy oil, or for applying hot liquid asphalt to road metal with penetrating pressures. When the road work is not pressing the iron tank can be lifted off and an ordinary body substituted.

A rotary pump, driven by a lay shaft from the rear of the truck transmission case and having 13-ft. lift, is used to fill the tank, to sprinkle or flush with water, to circulate road oils during heating, or to deliver oil to a set of 16 spreading nozzles in the rear. These nozzles spread uniformly over a strip of road 8 ft. 6 in. wide. Three sets are furnished for different work. Each throws a fan-like stream, one stream overlapping the other. If desired, the nozzles can be moved to the right or left of the center a distance of two feet. The nozzles and pump pressure are regulated by a man from a small seat in the rear and on the right side. This man can communicate with the driver of the truck, while the truck is in motion, by means of a double bell system. After the oil has been shut off the pump carries the quantity remaining in the pipes back into the tank and an automatic drip-trough falls, preventing any of the oil on the mouth of the nozzles from dripping to the road. This is particularly useful in passing over street crossings. After the truck has been used for oiling, the tank can be quickly cleaned by using the pump to circulate kerosene through it. Small tanks of kerosene and gasoline are located on the left side of the truck.

* * *

Hand-Thrown Derailing Block

A new hand-thrown derailing block has been put out by the Hobart-Allfree Co., Chicago. The device comprises a strong cast base and a pivoted arm with derailing head swinging over around the rail head. The base is reversible, and there are pivot-bolt holes at different levels so that the same block can be used with different sizes of rail. Registering padlock holes on base and arm permit locking in either clear or derailing position.